GEOTECHNIAL INVESTIGATION REPORT

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENT McKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

For

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1.0 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the proposed "Permanente Creek Flood Protection Project- Flood Protection Improvement McKelvey Park" project in Mountain View, California, hereinafter referred to as "PROJECT". The work was performed in general accordance with the scope of work outlined in our proposal to Hatch Mott MacDonald (Designer). The approximate location of the project site and its vicinity are shown on the Project Location Map, Plate 1.

The geotechnical recommendations presented in this report are intended for design input and are not intended to be used as specifications. In addition, the data provided in this report including these geotechnical recommendations should not be used for bidding purposes or for construction cost estimates. If the report is provided as a reference document, any interpretation of the data and recommendations should be the sole responsibility of the user and PARIKH Consultants, Inc. (PARIKH) shall not be liable for any consequences.

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general subsurface soil conditions at the project site, to evaluate their engineering properties, and to provide geotechnical recommendations for the foundation design of the retaining walls and geotechnical design of the pipeline of the proposed project.

The scope of work performed for this investigation included site reconnaissance; obtaining representative soil samples and logging soil materials encountered in the soil borings; laboratory testing of the collected soil samples, performing engineering analyses based on the field and laboratory data, and preparation of this geotechnical investigation report.

3.0 EXISTING FACILITITY AND PROPOSED CONSTRUCTION

Existing Facility

McKelvey Park is a 5-acre baseball facility owned and operated by the City of Mountain View.

The park is located at the corner of Park and Miramonte Avenue.

Proposed Construction

The following are the general description of the proposed construction of this project:

1. Approximately 74,000 cubic meters (60 acre-feet) of storage volume would be created in the 2 hectare (5 acre) site.
2. The site would be excavated approximately 22 feet deep, with vertical sides (supported by retaining walls) in order to maintain the baseball field dimensions as existing.
3. The parking lot area would be reconfigured such that it would slope down at 5 percent slope down to the new field level.
4. There would be stairs down to field level placed at various points along the perimeter, as well as two handicapped accessible ramps at Park Drive and Miramonte Avenue.
5. The inlet and outlet “wet well” would be located in the southwest corner of the property in the triangular parcel owned by the Santa Clara Valley Water District.
6. Pedestrian bridge near the southwest corner of the project site.

More details of the proposed Flood Protection Improvement Mckelvey Park Flood Detention are provided in the project plans.

Our recommendations presented in this report are based on the above information. Any major deviation should be reported to PARIKH for further consideration.

4.0 SITE CONDITIONS

Based on the topographic map provided by the designer, the topography in the vicinity of the project site is generally level within McKelvey Park. The ground surface elevations of Borings MK-B-1 through MK-B-7 vary from Elev. + 105.8 feet to Elev. +108.8 feet and the ground surface elevations of Borings MK-P-1 through MK-P-5 vary from Elev. +102.9 feet to Elev. +105.6 feet.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

The field exploration program was developed based on discussions with the designer and with review comments by the Santa Clara Valley Water District (SCVWD). A total of eleven borings (Borings MK-B-1 through MK-B-7 and MK-P-1 through MK-P-4) were drilled to the depths between 25 feet and 40 feet in June 2010. Borings MK-B-1 through MK-B-7 were drilled for the retaining walls required for the flood detention basin. Borings MK-P-1 through MK-P-4 were drilled for the 24-inch diameter reinforced concrete outlet pipe. This outlet pipe is no longer proposed for this project and a wet well will be provided to empty the basin at the southwest corner of the project site adjacent to the Permanente Creek according to the designer.

The approximate locations of the borings are shown in the “Site Plan”, Plate 2. The details of the field exploration are included in Appendix A. The descriptions of the materials encountered in the field exploration are shown on the "Log of Borings".

Laboratory tests were performed on selected soil samples collected during field exploration to evaluate the physical and engineering properties of the subsurface soils. The laboratory test methods and test results are presented on plates included in Appendix B. Laboratory test results for moisture content, total unit weight, unconfined compression, Plasticity Index and grain size classification of the soil samples are presented on the Log of Borings in Appendix A.

It should be noted that the descriptions of the soils encountered and relevant boring information presented on the Log of Borings depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the Log of Borings. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the boring locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the subsurface soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

6.0 SUBSURFACE SOIL CONDITIONS

Based on the boring data of Borings MK-B-1 through MK-B-7, the subsurface soil conditions encountered in the soil borings within the McKelvey Park are summarized in the table below.

TABLE 1A: SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	Boring Depth (ft)	Subsurface Soil Conditions
MK-B-1	30	Approximately 12 feet of medium stiff to very stiff lean clay, underlain by approximately 5 feet of medium dense sand, underlain by medium dense to very dense sand to the boring depth.
MK-B-2(W)	40	Approximately 13 feet of medium stiff to very stiff lean clay, underlain by very stiff lean clay with intermittent pocket/lens of very stiff silt to the boring depth.
MK-B-3	30	Approximately 3 feet of medium stiff lean clay, underlain by approximately 5 feet of very loose sand, underlain by approximately 5 feet of hard silt, underlain by stiff to very stiff lean clay to the boring depth.
MK-B-4	30	Approximately 8 feet of medium stiff lean clay, underlain by approximately 6 feet of loose sand, underlain by approximately 4 feet of very stiff silt, underlain by approximately 7 feet of very stiff lean clay, underlain by medium dense sand to the boring depth.
MK-B-5	30	Approximately 13 feet of soft to medium stiff lean clay, underlain by approximately 8 feet of loose sand, underlain by very stiff lean clay to the boring depth.
MK-B-6	30	Approximately 13 feet of stiff lean clay, underlain by approximately 5 feet of medium dense sand, underlain by approximately 5 feet of very stiff lean clay, underlain by approximately 5 feet of dense sand, underlain by very stiff lean clay to the boring depth.
MK-B-7	30	Approximately 22 feet of stiff to very stiff lean clay, underlain by approximately 5 feet of medium dense sand, underlain by stiff lean clay to the boring depth.

Based on the boring data of Borings MK-P-1 through MK-P-4, the subsurface soil conditions encountered in these soil borings are summarized in the table below

TABLE 1B: SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	**Boring Depth (ft)**	**Subsurface Soil Conditions**
MK-P-1	40	Approximately 12 feet of medium stiff to very stiff lean clay/silt, underlain by approximately 15 feet of medium dense sand, underlain by very stiff lean clay to the boring depth.
MK-P-2	40	Approximately 17 feet of medium stiff to stiff lean clay, underlain by interbedded layers of very stiff to hard silt/lean clay to the boring depth.
MK-P-3	40	Approximately 6 feet of loose sand, underlain by approximately 6 feet of dense sand, underlain by approximately 20 feet of very stiff lean clay, underlain by approximately 5 feet of medium dense sand, underlain by medium stiff to stiff lean clay to the boring depth.
MK-P-4	40	Approximately 7 feet of medium stiff lean clay, underlain by approximately 10 feet of medium dense sand, underlain by approximately 20 feet of stiff to very lean clay/silt, underlain by medium dense to dense sand to the boring depth.

Groundwater was measured at approximate depth of 39.5 feet in Boring MK-P-4. No groundwater was encountered in the other borings during drilling with the boring depth between 30 feet and 40 feet. Monitoring piezometer was installed in Boring MK-B-2 (W) in June 2010. Readings were taken by the designer from August 2010 through January 2012. Based on the monitoring record, no groundwater was measured from August 2010 through March 2011 and groundwater was measured at the depths between 39.2 feet and 40.2 feet in July 2011, October 2011 and January 2012.

Based on the “Seismic Hazard Zone Report – Mountain View 7.5’ Quadrangle Plate 1.2 – Groundwater”, the depth to historically high groundwater is estimated to be approximately 33 feet. Design groundwater elevation of +76 feet (approximately 33 feet from highest ground elevation of soil boring which is +108.8 feet of Boring MK-B-2(W)) is assumed for this project.

Groundwater level is anticipated to vary with the passage of time due to seasonal groundwater fluctuations, fluctuation of water elevations in the Permanente Creek, surface and subsurface flows, ground surface run-off, and other environmental factors, which may not be present at the time of the investigation.

7.0 GEOLOGY

The project site is in the southwestern portion of San Francisco Bay area in the Coast Range geomorphic province of California. The Coast Range forms a nearly continuous topographic barrier between the California coastline and the San Joaquin Valley. The Coast Range in this region is a double chain of mountain running north-northwest. Three prominent geologic blocks dominate the San Francisco Bay Area: the Santa Cruz Mountains (western block), the San Francisco Bay (central block), and the East Bay Hills/Diablo range (eastern block).

General geologic features pertaining to the project site and its vicinity were evaluated with reference to the Geologic Map titled "Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California", a digital database, compiled by E.J. Helley, R.W. Graymer, G.A. Phelps, P.K. Showalter, and C.M. Wentworth, May 1994. Based on the map, the project site is primarily underlain by Alluvial Fans and Fluvial Deposits (Qhaf) (Holocene) and Alluvial Fans and Fluvial Deposits (Qpaf) (Pleistocene). The general geology of the project area is shown on the Geologic Map, Plate 3. Descriptions of the main geologic units are as follows:

Qhaf - Alluvial Fans and Fluvial Deposits (Holocene): Unconsolidated, plastic, moderately to poorly sorted silt and clay rich in organic material. Seasonally saturated. Irregularly bedded.

Qpaf - Alluvial Fans and Fluvial Deposits (Pleistocene): Brown dense gravelly and clayey sand or clayey gravel that fines upward to sandy clay. They can be related to modern stream courses. They are distinguished from younger alluvial fans and fluvial deposits by higher topographic position. They are less permeable than Holocene deposits.

8.0 EARTHQUAKE CONSIDERATIONS

8.1 Seismic Sources

The project is located in a seismically active part of Northern California. Many faults exist in the regional area. These faults are capable of producing earthquakes and may cause strong ground shaking at the project site. Faults in the vicinity of the site with a moderate to high potential for releasing seismic energy include the Cascade Fault, San Andreas Fault Zone and Monte Vista-Shannon. The attached Fault Map (Plate 4) presents the locations of the fault systems relative to the project site.

Maximum moment magnitudes (Mmax) of some of the closest faults in the area are based on the Caltrans 2012 ARS Online Report. These maximum moment magnitudes represent the largest earthquake a fault is capable of generating and is related to the seismic moment. The earthquake data of the active faults in the project vicinity are summarized in the table below.

TABLE 2: EARTHQUAKE DATA

Fault (Fault ID)	Site-to-Fault Distance, Rrup* (km)	Maximum Moment Magnitude of Fault, M_{Max}
Cascade fault (153)	1.77	6.7
San Andreas (Peninsula) 2011 CFM (134)	10.45	8.0
San Andreas (Santa Cruz Mts) 2011 CFM (158)	12.81	8.0
Monte Vista-Shannon (154)	4.09	6.4
Silver Creek (148)	12.65	6.9
Hayward (South) (137)	18.89	7.3

*Closest distance (km) to the fault rupture plane

The Caltrans Fault Map (Plate 4) presents the locations of the fault systems relative to the project site.

8.2 Seismic Design Criteria

The recommended response spectrum for the bridge site was determined based on the Caltrans ARS Online (Ver. 2.3.06) for the development of response spectra for design. The development of the design Acceleration Response Spectrum (ARS) curve is based on several input parameters, including site location (longitude/latitude), average shear wave velocity for the top 30m/100 feet (V_{s30m}), and other site parameters, such as fault characteristics, site-to-fault distances.

The design methods incorporate both "Deterministic and Probabilistic Seismic Hazards" to produce the Design ARS. According to the recent Caltrans methodology, we have compared the Caltrans Probabilistic ARS with the ARS from 2008 USGS Deaggregation Hazard (beta) web site for the 5% in 50 years probability of exceedance (or 975 year return period) at periods of 0, 0.3, 1 and 3 seconds.

Typically, average shear wave velocity for the top 30m (100 feet) at the site is estimated by using established correlations and the procedure provided in the recent "Methodology for Developing Design Response Spectrum for use in Seismic Design Recommendations" (November 2012) publication by Caltrans. The average shear wave velocity is estimated to be 260 m/s.

The site location and the relevant parameters are summarized as follows, and the recommended design curve is presented on Plate 5A. Comparison of deterministic and probabilistic ARS curves is presented on Plate 5B.

1. Site Location: 37.3864ºN/122.0889ºW
2. Average V_{S30m}: 260 m/s
3. Basin Effect: No
4. Near Fault Effect: Yes
5. Anticipated Peak Ground Acceleration (PGA): 0.624 g
6. The recommended ARS curve is the "Caltrans Online Probabilistic ARS"
7. Maximum Moment Magnitude (M_{max}): 8.0

8.3 Seismic Hazards/Liquefaction Potential

Seismic Hazards

Potential seismic hazards may arise from three sources: surface fault rupture, ground shaking and liquefaction. Since no active faults pass through the site, the potential for fault rupture is relatively low. As shown on the Fault Map, Plate No. 4, the closest active fault is the Cascade Fault, which is located approximately 1.1 miles southwest from the project site. Based on available geological and seismic data, the possibility of the site to experience strong ground shaking may be considered high.

Liquefaction Potential

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction. Clays are generally not susceptible to liquefaction.

Based on the available boring information, the subsurface soil conditions at the project site consist of mainly medium stiff to very stiff lean clay/silt with intermittent layers of loose to dense sands and hard silt. Groundwater was only encountered in Boring MK-P-4 at approximate depth of 39.5 feet during drilling. The liquefaction potential at the project site is considered low.

9.0 FINDINGS AND RECOMMENDATIONS

9.1 General

This report was prepared specifically for the proposed project. Normal construction procedures were assumed throughout our analyses and represent one of the bases of recommendations presented herein. Our design criteria have been based upon the materials and conditions encountered in the soil borings. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

9.2 Earthwork and Grading

All grading operations should be performed in accordance with the project specifications and City of Mountain View standards. A qualified Geotechnical Engineer or the regulatory agency should observe the grading operation and perform moisture and density tests on prepared subgrade and compacted fill material. Any fill material imported to the project site should be non-expansive, relatively granular materials and should be reviewed by the Geotechnical Engineer.

Should there be any alternations of the proposed construction that will affect the stated bases of our recommendations, we should be informed so that PARIKH can review such changes and amend or submit additional recommendations.

Structure Backfill. The backfill behind the structures should be the Caltrans "Structure Backfill" (Caltrans Standard Specifications Section 19). The excavated soil material at the project site can be used as the structure backfill provided the soil material is tested to be conforming to the Caltrans' specifications for structure backfill.

The engineered fill or structure backfill should be compacted in lifts that do not exceed 8 inches in loose thickness. The compaction moisture content of the fill material should be within the range of 2-percent wet of laboratory optimum moisture content.

Areas to receive engineered fill or structure backfill should be excavated to remove any loose/soft soil materials. The resulting surface upon which fill is to be placed should be observed by the Geotechnical Engineer. Areas receiving engineered fill or structure backfill should be scarified; moisture conditioned and compacted in accordance with City of Mountain Standard Provisions.

Compaction of Fill and Subgrade. The project specific recommendations for required relative compaction as per ASTM D1557-91 are as follows:

- 90% for subgrade preparation, general fill and backfilling after removing trees, buried utilities, structures and depressions caused due to construction activities, etc.
- 90% for subgrade receiving fill, for material below 6 inches of the foundation subgrade and below 18 inches of the flexible pavement subgrade.
- 90% for selected backfill material
- 90% - 95% for all structure backfill behind retaining walls.

- Minimum 95% for upper 6 inches of foundation subgrade, Aggregate Base underneath the mat foundation and upper 18 inches of flexible pavement subgrade to reduce the potential for pavement failure.
- Minimum 95% for free draining pea gravel or clean crushed rock under slabs-on-grade.

PARIKH should review the final grading plans prior to grading to confirm that the intent of our recommendations is included in the plans.

9.3 Earth Retaining System

It is our understanding that the project will require construction of twelve retaining walls. Based on the "Permanente Creek Flood Protection Project – McKelvey Park Site Plan and Line Index", the following types of retaining wall are proposed for this project.

a) Type L Wall.
b) Caltrans Standard Type 1 Wall.
c) Sheet Pile Wall.
d) Soldier Pile Wall.

Information of approximate wall locations, type of walls, maximum wall heights and approximate length of the wall provided by the designer is summarized in Tables 3A and 3B.

TABLE 3A: SUMMARY OF EARTH RETAINING SYSTEM TYPE AND LOCATION

Wall No.	Wall List	Wall Type	Location (Along approx. Sta.)
1	Park Dr. Wall	Sheet Pile/Type 1	Between McKelvey Park and Park Drive and Miramonte Avenue
2	St. Joe's Wall	Type L/1	Southern boundary of Big Field
3	Creek Wall	Sheet Pile	Between Permanente Creek and Access Ramp
4	Truck Ramp Wall	Type 1	Between Access Ramp & planted terrace
5	Parking Wall No. 1	Type 1	Between Parking Lot & Big Field
6	Parking Wall No. 2	Type 1	Between Parking Lot & Big Field
7	Big Field Wall	Sheet Pile /Type 1	Northwest corner of Big Field
8	Big Field Stairs Wall	Type 1	Western boundary of Big Field
9	Little Field Wall	Type 1/L	Western boundary of Little Field
10	Little Field Ramp Wall No. 1	Type 1	Between Big Field and Little Field
11	Little Field Ramp Wall No. 2	Type 1	Between Big Field and Little Field
12	Little Field Stairs Wall	Type L	Southern boundary of Little Field

* Soldier pile wall is not shown but may still be used in future design.

TABLE 3B: SUMMARY OF EARTH RETAINING SYSTEM

Wall No.	Wall List	Maximum Height (ft)	Bottom of Footing Elev. (ft)	Approx. Total Length (ft)
1	Park Dr. Wall	22	+86.5	825
2	St. Joe's Wall	24	+81.1	495
3	Creek Wall	22	N/A	230
4	Truck Ramp Wall	24	+83.96	325
5	Parking Wall No. 1	10	+94.5	180
6	Parking Wall No. 2	10	+94.5	230
7	Big Field Wall	24	+85.05	350
8	Big Field Stairs Wall	24	+83.34	90
9	Little Field Wall	20	+84.85	360
10	Little Field Ramp Wall No. 1	16	+84.33	175
11	Little Field Ramp Wall No. 2	14	+90.87	90
12	Little Field Stairs Wall	20	+83.5	95

9.3.1 Type L Wall

This type of retaining wall is proposed along the perimeter of the "Big Field" and "Little Field" in the park. Based on the "Retaining Wall Types" prepared by the designer, Type L cantilever wall is a modification of the Caltrans Standard Type 1 wall and similarly constructed as the Type 1 wall.

The retained heights of this retaining wall range from 20 feet to 24 feet. Maximum retained height of 24 feet is assumed according to the designer. The footing elevations range from approximately +81.1 feet to +84.33 feet.

9.3.2 Caltrans Standard Type 1 Wall

This type of retaining wall is proposed along the perimeter of the “Big Field” and “Little Field” in the park and adjacent to the parking lot and access ramp. The retained heights of this retaining wall range from 10 feet to 24 feet. Maximum retained height of 24 feet is assumed according to the designer. The footing elevations range from approximately +81.85 feet to +90.87 feet.

9.3.3 Bearing Capacity for Caltrans Standard Type 1 Wall and Type L Wall

The following are the allowable bearing capacities (based on F.S. of 3) available for the foundation design of the Type L wall and Caltrans Standard Type 1 wall.

TABLE 3C: SUMMARY OF ALLOWABLE BEARING CAPACITY

Wall No.	Wall List	Toe Pressure (ksf)	Undrained Shear Strength/Friction Angle Used in Analyses (psf)	Allowable Bearing Capacity (ksf)	
				Dead Load	Dead Load + Live Load
				F.O.S. = 3.0	F.O.S. = 2.0
1	Park Dr. Wall	2.76	900 and 2250	2.90	4.30
2	St. Joe’s Wall	2.98	1750	3.30	4.90
4	Truck Ramp Wall	2.98	1750	3.30	4.90
5	Parking Wall No. 1	1.41	1750	3.30	4.90
6	Parking Wall No. 2	1.41	750	1.43	2.10
7	Big Field Wall	2.98	32-degree	5.90	8.90
8	Big Field Stairs Wall	2.98	1750	3.30	4.90
9	Little Field Wall	2.44	2000	3.70	5.60
10	Little Field Ramp Wall No. 1	2.73	2000	3.70	5.60
11	Little Field Ramp Wall No. 2	1.95	30-degree	3.10	4.70
12	Little Field Stairs Wall	2.73	1500	2.80	4.20

A Factor of Safety of 3.0 is applied for dead load only and a Factor of Safety of 2.0 is applied to dead load plus live load. The above allowable bearing capacities for dead plus live loads may be increased by one-third for short term seismic and wind loads.

9.3.4 Sheet Pile Wall/Soldier Pile Wall

This type of retaining wall is proposed at the locations as listed in Table 3A. Maximum retained height of 24 feet is assumed according to the designer. Based on the available boring data in the vicinity of the proposed sheet pile wall/soldier pile walls, the subsurface soil material behind the proposed soldier pile walls may consist of mainly firm to stiff lean clays.

- Design Wall Height = Maximum Retained Height (feet)
- Unit Weight = 125 lb/ft^3 (pcf)
- Traffic Surcharge = 240 lb/ft^3 (psf)
- Active Earth Pressure: Ka = 0.31 (ϕ =32° for native soil)
- At-Rest Earth Pressure: Ko = 0.70
- Passive Earth Pressure: Kp = 5.84

For steel pile wall, the passive resistance with 672 pcf Equivalent Fluid Pressure (EFP) (above groundwater) with a maximum value of 6,720 psf (above groundwater) below the base of excavation may be provided by the soil resistance. These are the ultimate values of the passive soil resistance. Usually for sheet pile wall design, a factor of safety of 1.5 is adopted.

Soldier Pile Wall

- Arching capability = 2.0
- f(arching factor per Caltrans Trenching & Shoring Manual) = 1.0 assuming 36-inch diameter soldier piles with pile spacing of 6 feet on centers

Self-compacting material is recommended for the wall backfill if required. Geocomposite drain with weep hole should be placed between the lagging and the shotcrete facing.

9.3.5 Compacted Impervious Soils behind Retaining Wall

According to the designer, portion of the backfill behind the retaining wall will be paved. The upper two feet of the backfill behind the retaining wall which will not be paved is recommended to be placed with compacted imperious clayey soils to prevent surface water runoff and/or rainfall infiltration from seeping into the structure backfill behind the retaining wall.

9.4 Buildings

Based on the 60% Design Plan, three buildings are proposed for this project. It is our understanding that these buildings will be supported on the compacted structure backfill behind the retaining wall. The surcharge from the building should be considered in the retaining wall design. The name of the building, the retaining wall supporting these buildings and the "Finish Floor Elevation" are summarized in the table below.

TABLE 4: SUMMARY OF BUILDING FOUNDATIONS

Building Name	Supported on Retaining Wall No.	Finish Floor Elevation (ft)
Babe Ruth Field Building	RW-7 (Big Field Wall)	106.0
Babe Ruth Field Community Building	RW-11 (Little Field Ramp Wall)	106.0
Little League Field Building	RW-9 (Little Field Wall)	107.0

9.4.1 Foundation Design for Buildings

Based on the available boring information of Boring MK-P-1 (Babe Ruth Field Building) and Boring MK-B-6 (Babe Ruth Field Community Building and Little League Field Building), the subsurface soil conditions at the foundation subgrade generally consist of firm to stiff lean clay/sandy lean clay.

The buildings can be supported on a conventional spread footing foundation established on properly compacted structure backfill.

9.4.2 Bearing Capacity for Buildings

Calculated allowable bearing capacities for various buildings, which may be used for the design with respect to dead load and dead-plus-live-loads, are summarized in the table below.

TABLE 5: RECOMMENDED BEARING CAPACITY FOR BUILDINGS

Building Name	**Recommended Bearing Capacity Supported on Fill (ksf)(1)**	
	Dead Load	**Dead Load + Live Load**
Babe Ruth Field Building	7.0	10.5
Babe Ruth Field Community Building	7.0	10.5
Little League Field Building	7.0	10.5

1. Bottom of footing assumed at a minimum depth of 2 feet below adjacent finish grade.

The above allowable bearing capacities for dead plus live loads may be increased by one-third for short term seismic and wind loads.

9.4.3 Modulus of Vertical Subgrade Reaction for Buildings

The mat foundation should have a minimum thickness of 12 inches. For these conditions, the recommended allowable bearing capacities should be as shown in Table 5. The mat foundation may be designed for a modulus of vertical subgrade reaction summarized in the table below.

TABLE 6: RECOMMENDED MODULUS OF VERTICAL SUBGRADE REACTION

Building Name	**Modulus of Vertical Subgrade Reaction for Compacted Structure Backfill (pci)**
Babe Ruth Field Building	146
Babe Ruth Field Community Building	146
Little League Field Building	146

The mat foundation should be underlain with 6-inch of ¾-inch crushed rock.

9.4.4 Concrete Slabs-on-grade

The backfill against the slab should be compacted and graded to shed away from the perimeter using non-expansive soil. If moisture migration through the slabs is undesirable, a moisture barrier and capillary break may be provided between the slab and the compacted subgrade. The capillary break consists of 5 inches of free draining pea gravel or clean crushed rock. A moisture barrier is generally an imperious membrane, which should be placed over the capillary break. The membrane should be covered with a 2-inch thick layer of sand to protect it during construction. The sand should be slightly moist just prior to pouring the slab to aid in curing the concrete.

Minor cracking and maintenance problems may be anticipated for slab-on-grade construction; however, separation from structure with concentrated loads and keeping the subgrade moist during construction can greatly reduce the amount of eventual cracking. Prior to concrete placement, any dry cracked subgrade should be sealed with appropriate watering or should be excavated and re-compacted. These conditions and construction procedures should be observed by the Geotechnical Engineer.

9.4.5 Seismic Design Criteria for Buildings

It is our understanding that seismic design of the project will follow the 2013 California Building Code (CBC). Based on the subsurface data, it is our opinion that the subsoil can be categorized as Soil Type D (“Stiff Soil”) per NEHRP criteria. The Soil Type D is determined based on boring drilled within the project area. The shear wave velocity for individual layers were estimated based on the formulations given in Caltrans Geotechnical Services Design Manual (August, 2009) on Seismic Parameters. The average shear wave velocity for the upper 100 ft and seismic soil type were estimated in accordance with CBC 2013.

Based on Chapter 16 of the 2013 California Building Code (CBC) and existing boring data, the relevant seismic coefficients per 2013 CBC seismic criteria are shown as follows:

Table 7: Seismic Design Criteria per CBC (2013 California Building Code)

Coefficient	Recommended Value
Site Coefficient, F_a (Table 1613A.3.3 (1))	1.00
Site Coefficient, F_v (Table 1613A.3.3 (2))	1.50
S_S	1.53
S_1	0.67
$S_{MS} = F_a \times S_S$ (Equation 16A-37)	1.53
$S_{M1} = F_v \times S_1$ (Equation 16-38)	1.00
$S_{DS} = 2/3 \times S_{MS}$ (Equation 16-39)	1.02
$S_{D1} = 2/3 \times S_{M1}$ (Equation 16-40)	0.67

*Approximate Site Coordinate = 37.3864°N 122.0889°W

9.4.6 Foundation Drainage for Buildings

The following can be considered for the foundation drainage of the three buildings:

a) The final exterior grade adjacent to the proposed structure should be such that the surface drainage will flow away from the structure. Rainwater discharge at downspouts should be directed onto pavement sections, splash blocks, or other acceptable facilities, which will prevent water from collecting in the soil adjacent to the foundations.

b) Utility lines that cross under or through slab, footings, or wells should be completely sealed or waterproofed, as necessary, to prevent moisture intrusion into the areas under the slab and footings.

9.5 Lateral Earth Pressures

Retaining walls and thrust blocks should be designed to resist the following Lateral Earth Pressures (Equivalent Fluid Pressures-EFP) and live load. The “above water” values assume no hydrostatic pore pressure buildup behind the wall and are based on the assumptions that drainage measures are provided behind the walls.

For the lateral design of the retaining wall structures, the following lateral earth pressures (Equivalent Fluid Pressures) and design parameters should be used:

TABLE 8: SUMMARY OF LATERAL EARTH PRESSURES

Design Condition	**Native (Cohesive) Soil (Assume Ø = 32°)**		**Structure Backfill (Assume Ø = 32°)**	
	Above water	Below water	Above water	Below water
Active	39.0 pcf	21.0 pcf	39.0 pcf	21.0 pcf
At-rest	87.5 pcf	47.0 pcf	59.0 pcf	32.0 pcf

(a) The available ultimate passive resistance against the side of the footing is 672 pcf EFP above water and 395 pcf EFP below water. According to Caltrans Bridge Design Specifications (August 2003) Section 5 (5-47), not more than 50% of the available passive lateral earth pressure shall be considered in the determination of the factor of safety against sliding (only for cantilever retaining wall when combining passive and bottom friction).

(b) A hydrostatic pressure of 62.4 pcf EFP should be applied to the below water case.

(c) A coefficient of friction of 0.34 may be used to estimate the frictional resistance along the bottom of the footing. Only the dead loads should be used to estimate the frictional resistance at the bottom of footings.

Cantilever walls, which are free to rotate at least 0.01 radii, may be assumed flexible and designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead, live or traffic load) should be added to the preceding lateral earth pressures. A coefficient of 0.31 and 0.47 (for native soils) may be used to determine the additional earth pressure resulting from the surcharge for cantilever walls (active) and rigid (at-rest) walls, respectively. When traffic is anticipated within a horizontal distance of one half of the wall height, Caltrans design practice is to add an equivalent earth pressure of 240 psf.

9.5.1 Seismic Lateral Earth Pressure for Retaining Walls

The proposed retaining walls will experience increased lateral loads during earthquake shaking. The design needs to consider seismic event per the AASHTO LRFD (Sections 11.9.6 & 11.8.6). The additional horizontal forces recommended to simulate earthquake loads are dependent upon the magnitude of ground surface accelerations and the retained height of the retaining wall, together with the weight and type of material retained by the retaining walls. In general, the pseudo-static approach developed by Mononobe and Okabe (M-O) may be used to estimate the equivalent static force using a seismic coefficient K_h = 0.5 Amax. According to Appendix A11.1.1.1 (California Amendment), the seismic incremental lateral earth pressure is assumed to have triangular distribution over the wall height. Per new Caltrans ARS Online, the anticipated Peak Ground Acceleration (PGA) at the site is about 0.624 g. Therefore, a K_h value of 0.312 g was used to evaluate the seismic lateral earth pressure.

For this site, we have adopted a unit weight of γ = 125 pcf. Based on the analyses, incremental seismic lateral force with a value of 15.0 H^2 (in lb per linear foot of wall) for the flexible walls and 25.5 H^2 (in lb per linear foot of wall) for the restrained walls are recommended. The resultant force from the incremental seismic lateral earth pressure is triangularly distributed acting one-third of the wall height (H in ft) per California Amendments.

The earth pressures for shoring design should be referred to Section 10.1.2 *"Selection of Shoring System"* of this report.

9.6 Foundation Design of Pedestrian Bridge

According to the "Permanente Creek Flood Protection Project – McKelvey Park Site Plan and Line Index", a pedestrian bridge is proposed with the south abutment near the wet well and the north abutment near the concrete pedestrian path. The pedestrian bridge will be supported on spread footing according to the designer.

9.6.1 Bearing Capacity for Pedestrian Bridge

The footing elevation for the abutments of the pedestrian bridge will be Elev. +84.4 feet according to the designer. An allowable bearing capacity (based on

F.S. of 3) of 3.3 ksf is available for the foundation design of the pedestrian bridge abutment at this footing elevation.

9.6.2 Ultimate Seismic Passive Resistance for Pedestrian Bridge

Passive resistance of 5.0 ksf (ultimate) is assumed for seismic design of the abutment backwall (5.5 feet or greater). For activated height “H” less than 5.5 feet, modify the passive resistance proportionally i.e. passive resistance is equal to 5.0x (H/5.5) ksf. A minimum lateral wall movement of 2% of wall height to mobilize the full ultimate passive resistance is required.

9.7 Wet Well

A wet well is proposed to empty the basin at the southwest corner of the project site adjacent to the Permanente Creek. The size of the footing is anticipated to be 12 feet x 12 feet according to the designer.

9.7.1 Foundation

Foundations for buried structures such as the wet well may be supported on a conventional mat-type foundation established on properly compacted fill in a dry subgrade condition (to prevent sand boiling and clay heaving, etc.). The foundation should be underlain by at least 2 feet of properly compacted Class 2 Aggregate Base. The limits of this base should extend a minimum of 2 feet beyond the projected foundation lines. The Class 2 Aggregate Base should be as per Caltrans Standard Specifications and should be compacted to not less than 95 percent relative compaction of maximum dry density as per project specifications.

Typically, the design of mat foundations models the mat/soil interaction as a flexible mat on elastic foundation. The recommended modulus of subgrade reaction of native soils for analysis is 70 pci. The estimated settlement for foundations constructed as recommended above is anticipated to be within tolerable limits for the proposed structures.

The subgrade for the mat foundation (i.e. native soil below the Class 2 Aggregate Base) should be prepared by scarifying up to 6 inches and re-compacting to not less than 95 percent relative compaction of maximum dry density as per project specifications.

9.7.2 Design for Uplift Pressures

Based on the groundwater encountered in the field explorations, the proposed wet well may be subject to submerged or partially submerged conditions. Buoyancy should be considered for the design.

The buoyant uplift force acting on the proposed wet well may be resisted by a combination of the dead weight of the structures and the friction between the structure walls and the adjacent soils. To estimate the frictional resistance, a coefficient of friction of 0.35 is recommended between the walls and the adjacent structure backfill. The horizontal pressure acting on the wall may be calculated as 0.36 times the corresponding vertical pressure at depth. For design purposes, the vertical pressure at depth may be estimated using soil unit weight of 125 pcf and 68 pcf above and below the design groundwater level of +76 feet, respectively.

In the event that additional uplift resistance is needed, the foundation may have to be enlarged beyond the walls to engage additional weight of the structure backfill.

9.7.3 Lateral Design

For the lateral design of the wet well, the lateral earth pressures (Equivalent Fluid Pressures) and design parameters recommended in Section 9.4 should be used.

Refers to Section 10.1.2 *"Selection of Shoring System"* below for shoring construction.

9.8 Subgrade Preparation

The subgrade should be prepared to provide uniform bearing throughout the entire area of the structure. Subgrade shall be smooth, uniform, and free of debris, sharp rocks. Where hard unyielding materials larger than 3 inches, such as gravels, cobbles, etc., which may cause local stresses are encountered, they should be excavated out and backfilled with structure backfill.

Generally, the materials anticipated at the foundation subgrade of the proposed retaining walls, wet well and buildings primarily consist of medium stiff to very stiff clay and/or loose to medium dense sand as indicated from the field exploration. In the event local soft soil conditions are encountered, it may be necessary to stabilize the subgrade prior to placing of the footing. Stabilization can be accomplished by over-excavating the subgrade and replacing with a pad. The pad could be constructed as follows:

(a) Over-excavate approximately 2 feet of existing soil below the footing subgrade. The over-excavation should extend 2 feet beyond the footing footprint.

(b) A layer of high strength geofabric (Mirafi 600X or equivalent) should be placed over the over-excavated subgrade.

(c) Place 2 feet of Class 2 Permeable 3/4-inch crushed rock over the geofabric.

(d) The end of geofabric should be overlapped on top of the Class 2 Permeable 3/4-inch crushed rock for a minimum distance of 2 feet.

The imported crushed rock and sources should be approved by the Engineer or the Agency prior to importing and placing the materials on the project site. The top of the working platform may be densified by vibratory compaction equipment prior to placing the concrete footing. The Contractor's method of compacting the working platform should be reviewed by the Engineer. The Contractor may use other methods of subgrade stabilization. The Contractor's proposed method should be reviewed by the Engineer.

The actual locations and quantity of the over-excavations may vary depending on the subsurface soil conditions in the field. The Geotechnical Engineer should review the over-excavations during construction. It may be prudent to have these available on a unit rate or contingency basis. If shoring is used during construction, it should be designed to accommodate the above discussed over-excavation depth.

9.9 Utility Relocation

Utilities mainly in the vicinity of Miramonte Avenue and Park Drive will be relocated with the following details:

a) Existing 8-inch CIP W will be removed and 8-inch PVC W will be relocated and constructed.
b) Existing 8-inch VCP SS will be abandoned and 12-inch PVC SS will be relocated and constructed.
c) Existing 33-inch RCP SD will be abandoned, relocated and constructed.
d) Existing 54-inch RCP SD will be abandoned and 60-inch RCP SD relocated and constructed.

The length, pipe invert and the approximate depth of the proposed relocated pipe are summarized in the table below:

TABLE 9: SUMMARY OF RELOCATED PIPES

Relocated Pipe	Length (ft)	Pipe Invert (ft)	Approx. Depth (ft)	Pipe Invert (ft)	Approx. Depth (ft)
8” PVC W	289	102.0	4.0	101.2	3.6
12” PVC SS	253	100.0	6.0	99.0	5.8
33” RCP SD	138	97.8	7.1	96.7	8.6
60” RCP SD	55	94.0	10.8	95.9	11.0

All the trench excavation, backfill and resurfacing for the design and construction of the relocated utilities should be in accordance with “City of Mountain View Public Works Standard Provision (CMV Standard Provision) Section 24: TRENCH EXCAVATION, BACKFILL AND RESURFACING” and “City of Mountain View Department of Public Works Standard Detail Trench Paving, Backfill and Pipe Bedding Sections” (CMV Standard Details).

9.9.1 Thrust Blocks

At locations where the pipeline changes direction, dead ends or develops thrusts at valves or other attachments, the tendency of movements can be resisted by (a) friction between the pipe and surrounding backfill, and (b) thrust blocks. A coefficient of 0.35 can be used to determine the friction between the pipe and the backfill. Thrust blocks can be designed for the lateral earth pressures as described in Section 9.4 above.

Thrust blocks should be at least 12 inches wide and should be founded at least 12 inches below the pipe invert. In sizing the thrust blocks for design, it should be noted that the maximum lateral pressures of the thrust block should not exceed the allowable bearing capacity of 1.5 ksf.

Other restrained joint systems for watermain in accordance with CMV standards are also acceptable.

9.9.2 Bedding Subgrade

In general, the subsurface soil conditions encountered at the planned pipe invert depth (approximately 4 to 11 feet) of the relocated utilities consist of soft to very stiff clay or loose to medium dense sand. The subgrade should be prepared to provide uniform bearing throughout the entire length of the pipe alignment.

When the trench bottom is unstable due to a wet or spongy foundation, the trench bottom must be stabilized in accordance with CMV Standard Provision Section 24-03.03.

9.9.3 Pipe Bedding

The pipe bedding and bedding material details should conform to the CMV Standard Provision Section 24-03.03, which consists “Initial Backfill (Selected Backfill Material)” and CMV Standard Details.

Selected Backfill Material

According to CMV Standard Provision Section 24-02.01, the selected backfill material shall be sand or granular material. The select backfill material shall have a size and gradation falling within the following limits:

Sieve Sizes	Percent Passing
1/2”	100
No. 4	50 – 100
No. 200	15 Maximum

The minus two hundred (200) portion of the material expressed as a percentage multiplied by the Plasticity Index shall not exceed one hundred (100).

Most of the materials anticipated within the planned depth of excavation for the proposed project are relatively fine-grained clayey material and are not recommended for pipe bedding. The material should be reviewed by the Engineer prior to importing to the site.

The bedding material will be generally more permeable than the surrounding native soils and may provide a conduit for the flow of groundwater. If it is desired to minimize this flow, it is our opinion that cut-off dams be placed along the pipeline at frequent intervals. The spacing of the cut-off dams is a function of the accepted risk. Cut-off dams could consist of compacted native clay soils, concrete, soil-cement or cement-bentonite-water slurry. The soil-cement and cement-bentonite-water slurry used for cut-off dams should be designed for a permeability not to exceed 10^{-7}cm/sec.

The thickness of bedding material should be minimum 6 inches. The Contractor should limit both method and equipment so as not to damage the integrity of the pipe or related facilities or induce a diametrical variation of greater than plus or minus 0.5%.

Controlled Density Fill (CDF)

CDF shall consist of a fluid, workable mixture of aggregate, cement and water. CDF mix shall be proportioned to be a flowable, non-segregating, and self-consolidating, low-shrink slurry with an unconfined compressive strength at 28 days shall be a minimum 150 pounds per square inch (psi). After setting, the unit weight of the CDF shall not exceed 115 pounds per cubic foot (pcf).

CDF cannot be used for initial backfill unless it has been approved by the Engineer as a backfill material according to CMV Standard Provision Section 24-04.01. The use and the details of CDF should be in accordance with CMV Standard Provision 24-02.04.

9.9.4 Trench Backfill

The material for the trench backfill, depth of initial backfill and compaction of the trench backfill etc. should be in accordance with CMV Standard Provision Section 24-04 “TRENCH BACKFILL”. The compaction of the trench backfill should be in accordance with ASTM D1557-91.

The backfill materials should be placed by methods that do not disturb or damage the pipes. Within the pipe zone, heavy equipment operating over the pipe could result in damage, excessive deflections or other disturbances to the pipe where soil cover is insufficient. A minimum soil cover of 3 feet should be placed over the crown of the pipe prior to the use of heavy equipment.

Since the native materials are primarily fine-grained and cohesive, Jetting or flooding should not be permitted for backfill compaction.

The pipe bedding and the trench backfill are recommended to be wrapped around by engineering fabric to prevent the migration of fines from the native soil materials to the pipe bedding and the trench backfill. The engineering fabric could be either woven or non-woven type fabric and should conform to Caltrans Standard Specification (Section 88-1.04, Type B).

9.9.5 Buoyancy of Pipes

Groundwater was generally not encountered during the field explorations in June 2010. The depth to the pipe invert ranges from approximately 4 feet to 11 feet. The minimum soil cover ranges from approximately 3 feet for the 8-inch PVC pipes to approximately 5 feet for the 33-inch RCP pipe. Therefore, the proposed relocated pipeline is unlikely to be subjected to submerged or partially submerged conditions under normal condition. However, buoyancy should be considered for the pipes at the bottom of the basin that will be submerged in a flood event.

The buoyancy of a pipeline depends upon the weight of the pipe, the weight of the volume of water displaced by the pipe, the weight of the liquid load carried by the pipe and the weight of the backfill. As a conservative analytical practice, consider the pipeline empty so the weight of the liquid will be considered as an additional safety factor and to allow for the possibility of the pipeline not being in use during a period of time.

The design of buoyancy of pipeline is to ensure the total weight of the pipe and the weight of the backfill (both inundated and above the level of inundation) above the pipe is greater than the weight of the volume of water displaced by the pipe. A safety of 1.5 is normally applied to the weight of the backfill. For design purposes, the vertical pressure at depth may be estimated using soil unit weight of 125 pcf and 68 pcf above and below the level of inundation, respectively.

The following can be considered to prevent flotation in the event the weight of the pipe and backfill is not adequate:

a) Increase the wall thickness of pipe or change the pipe to a heavier pipe material;
b) Design concrete collars on the pipe. Submerged weight of concrete should be used when computing the volume of concrete per linear foot pipe anchorage. Submerged unit weigh of concrete is 87.6 pounds per cubic foot.

9.10 Shrinkage Factor for Earthwork

Shrinkage is the percentage of soil decrease in volume when it is compacted. Shrink and swell percentage generally have a wider range for sands and rocky, gravely soils than for clays. The excavated material from the project site is anticipated to be mostly cohesive soils such as clay and silt. Typical Shrinkage Factor of 0.9 can be assumed for the cohesive soils.

9.11 Corrosion

The corrosion investigation for this project was performed in general accordance with the provisions of California Test Method 643. Chemical tests were performed on soil samples from Borings MK-B-5 and MK-P-2 to evaluate the corrosion potential of the subsurface soil. A summary of the corrosion test results is presented in the following table.

TABLE 11: SUMMARY OF CORROSION TEST RESULT

Boring	Minimum Resistivity (ohms-cm)	PH	Water-soluble Chloride (ppm)	Water-soluble Sulfate (ppm)
MK-B-5	2,360	7.78	4.3	4.4
MK-P-2	1,690	7.75	36.9	49.6

According to Caltrans Corrosion Guidelines, November 2012 (Version 2.0), Caltrans considers a site to be corrosive to foundation element if one of the following conditions exists for the representative soil samples taken at the site:

- Chloride concentration is greater than or equal to 500 ppm,
- Sulfate concentration is greater than or equal to 2000 ppm,
- pH is 5.5 or less.

Based on the corrosion test results, the native subsurface soils in Borings MK-B-5 and MK-P-2 are considered non-corrosive per Caltrans corrosion design guidelines. Standard Type II modified or Type I-P (MS) modified cement may be used for the concrete substructure.

10.0 CONSTRUCTION CONSIDERATIONS

The following sections of the report include comments related to excavation, dewatering, temporary excavation and shoring, foundation construction, earthwork and other geotechnical aspects of the proposed construction. The sections are written primarily for the engineer responsible for the preparation of plans and specifications. Since these sections discuss potential construction issues related to the project, it may also be of use to the City of Mountain View's representatives involved in monitoring of construction activity. The field investigation performed by PAIKH primarily address design issues and was not intended to identify construction-related issues.

Prospective contractors for the project must evaluate construction-related issues on the basis of their own knowledge and experience in the local area, on the basis of similar projects in other localities, or on the basis of field investigation on the site performed by them, taking into account their own proposed construction methods and procedures. In addition, construction activities related to excavation and lateral earth support must conform to safety requirements of OSHA and other applicable municipal and State regulatory agencies.

10.1 Temporary Excavations Slope and Shoring

Excavations should not be expected to stand vertically without any support. According to OSHA Safety Standards, temporary excavations with personnel working within the excavations should be sloped or shored if the excavations are deeper than 5 feet. Temporary slopes may be constructed if there is no site constraint. Temporary shoring should be used if site constraints are present or at the Contractor's option. It should be noted that the slope ratios recommended by OSHA are for temporary, un-surcharged slopes. Traffic and surcharge loads should be kept back at least 15 feet from the top of the excavations unless they are accounted for in the design of the support system. In addition, it should also be noted that the recommended slopes are for adequately dewatered excavations. All temporary excavations should be made and supported in accordance with California OSHA Safety Standards and should be designed based on the existing soil/site condition. "Bracing and Shoring" should be in accordance with CMV Standard Provision Section 24-03.06

In our opinion, conventional equipment could be used to excavate on-site soil materials. The materials to be excavated are anticipated to be mainly stiff to very stiff clays and loose to medium dense sands. It is possible that unknown old buried utilities or abandoned structures, concrete rubble etc. are located along the alignment. It might require special equipment and additional efforts to remove these buried objects.

All excavations should be closely monitored during excavation/construction to detect any evidence of instability, soil creep, settlements, etc. Appropriate mitigation measures and a comprehensive monitoring plan should be implemented to correct such situations that may cause or lead to future damage to residence/improvements, existing utilities and streets. The project specific crack survey and construction monitoring program should be implemented and is critical to this project. This can alert the Contractor and/or City of Mountain View to minimize potential risks and to allow implementation of mitigation measures (if any).

Groundwater was not encountered during the field explorations. However, groundwater may be fluctuated and expected for excavations extending below the groundwater level. A properly designed and constructed dewatering operation is required regardless of the construction method used. The groundwater should be maintained at least 3 feet below the bottom of the excavation at all times. The issues related to “dewatering” are discussed in Section 10.4 below.

Operation of construction equipment and the resulting vibrations may adversely affect the native soils at the site. This should be taken into consideration in the evaluation of temporary slope stability and shoring system.

10.1.1 Temporary Excavation Slopes

The slope height, inclination, and excavation depths should not exceed those specified in local, state, or federal safety regulations. The design of the temporary slopes by the Contractor or his specialty subcontractor should conform to the OSHA’s “Guidelines for Excavations and Temporary Sloping”. The Contractor or responsible subcontractor should develop their design based on the subsurface conditions exposed at the time of construction.

For excavations in homogenous soils 20 feet or less in depth, OSHA guidelines state that the maximum allowable slope should be 3/4H: 1V, 1H:1V and 1-1/2H:1V for Type A, B and C soils, respectively. (In general, Type A soils are stronger; Type B soils are intermediate, and Type C soils are weaker.) Based on our evaluation of the materials encountered in our borings, all un-shored slopes less than 20 feet deep should be excavated to inclination no steep than 1-1/2H:1V unless shored with applicable safety standard. It should be noted that the slope ratios recommended by OSHA are for temporary, un-surcharged slopes. Traffic and surcharge loads should be set back at least 15 feet from the top of the temporary slopes. Slopes may need to be flattened based on materials exposed during construction. Exposed slopes should be kept moist (but not saturated) during construction. The temporary cut slopes discussed above assume that the groundwater is maintained below the bottom of excavation at all time during construction.

Fills to be placed on temporary slope should be keyed and benched into the slope. The height of the key should not be less than 3 feet or more than 5 feet and the minimum width should be 6 feet. The frequency and location of the benches may be adjusted and modified in the field as necessary.

Strength softening, sloughing and erosion could be expected for the bare surficial soil materials if the temporary slopes are exposed to weather and rain for an extended period of time. Stiff clays also tend to develop soil creep due to seasonal change in moisture content resulting in sloughing. Therefore, adequate surface protection should be provided to protect the slope surface from erosion, excessive drying and/or saturation during construction.

10.1.2 Selection of Shoring System

If the temporary slopes extend beyond the right-of-way, undercut any existing buried utilities, foundations of adjacent structures and trees (i.e. excavation is within a 1-1/2H:1V slope projected downward from the bottom edge of adjacent footing), shoring may be necessary for the excavations. The selection, design and performance of the temporary shoring system and safety of construction personnel should be the responsibility of the Contractor or his specialty subcontractor. The Contractor should have the shoring system designed and signed by a Registered Engineer. The Contractor should evaluate the actual site conditions and select appropriate construction methods. Trenching and shoring operations should be conducted in accordance with applicable Cal OSHA codes.

The shoring system should be designed to be relatively rigid and with as many supports or struts as necessary to prevent excessive straining and deformation of the supported soils. This is also important with regard to existing asphalt concrete pavements where tension cracking may develop, even under minor strains. It is also very important with regard to protection and movement of the existing utilities.

The designed shoring systems considered may include bracing system (such as "speed shoring") with continuous support of the sidewalls, continuous interlocking sheet pile, or combination of the above depending on specific site conditions. A properly designed and constructed dewatering operation is required regardless the construction method used. Trench shield (boxes) or similar passive shoring systems are not recommended as exclusive methods of support unless it can be demonstrated through field monitoring (inclinometer) that lateral soil creep will not affect existing facilities and utilities. Continuous interlocking sheet pile is recommended for relatively deep excavations below the groundwater level with trench bottom stability concerns. The selection of construction method, shoring system and their design and performance for the project should be the Contractor's responsibility.

The sheeting should be driven to a sufficient depth below the excavation to prevent trench bottom instability (bottom heave, piping and blow out). The required depth should be determined by the Contractor/shoring designer. The type of driving equipment (vibratory or impact hammers) employed for sheet pile installation should be chosen and verified for drivability by the shoring contractor. "Unconventional" method of driving such as banging with backhoe bucket is not recommended. The temporary sheeting should be installed on both sides of the alignment in advance of the excavation. Horizontal struts should be placed against both sides of the sheeting at regular depths as the walls are exposed in order to maintain continuous stability of the excavation.

Bracings should also be installed as soon as practical against the continuous sidewall support. Failure to provide such struts/bracings in a timely manner may result in lateral creep, which may cause damage to existing facilities. Removal of sheeting should be carried out in a manner that does not adversely affect the bedding of the pipe or any other improvements. The opening after removal of sheeting should be filled with grout.

Earth Pressures for Shoring Design

Lateral earth pressure recommended for the design of sheeting or shoring systems is shown in Plate 6. For above groundwater conditions, a trapezoidal pressure diagram is recommended. When groundwater is present, the hydrostatic (triangular) pressure should be considered in addition to earth pressure (see Plate 6).

Any traffic and surcharge loads (such as stockpile material) should be set back at least 15 feet from the top of excavations. Otherwise, these loads should be considered in the shoring design. An additional height with equivalent thickness of 2 feet of native soil is added to the ground profile to determine the additional earth pressure resulting from the normal traffic load. The moist unit weight of the native soil is assumed to be 125 pcf. Such loads should be added to the horizontal loads at the shoring system at a rate of 40 % of the total applied load.

It is also assumed that the groundwater level within the trench will be lowered to about 3 feet below the excavation. For a contractor-designed shoring system, the Contractor should be encouraged to verify the soil conditions in order to satisfy their design requirements and to provide adequate shoring for the project.

Ground Settlement and Lateral Movement due to Shoring Deflection

Local experience with similar soil conditions, as well as published data, indicates that settlement of the ground surface adjacent to the shoring commonly is on the order of 0.25% to 0.5% of the trench depth. Lateral movement of same order of the magnitude should also be anticipated. This assumes good workmanship and the preloading of struts by onward jacking against the excavation sides to reduce lateral movement.

The settlement diminishes in magnitude with distance away from the shoring. Typically, the zone of influence of the maximum settlement has been found to extend to a distance of about 0.7 times the excavation depth away from the shoring and then diminishes in magnitude gradually to zero at a horizontal distance of about 2 to 2.5 times the depth. If existing utility lines are located within the potential zone of influence, they could experience distress as a result of the movement.

10.2 Excavation Bottom Stability

Due to fluctuation in groundwater elevation, excavation bottom instability during excavation may occur as a result of bottom heave, piping, or blow-out. Bottom heave is typical for excavations in clays. Deep shoring system and construction dewatering may be required to mitigate the condition.

10.3 Working Platform

If groundwater is encountered during excavation, soft and loose, saturated native soil deposits may be encountered at the excavation bottom. In such cases working conditions at the bottom of the excavation may become difficult; equipment used at the bottom of the excavation may lose mobility, etc. The Contractor should take adequate measures to minimize the disturbance of the sensitive deposits at the excavation subgrade. The Contractor may minimize the disturbance of sensitive deposits or mitigate existing soft ground conditions by constructing a working platform similar to the pad as described in Section 9.7 above.

The Contractor may use other methods of subgrade stabilization. The Contractor's proposed method should be reviewed by the Geotechnical Engineer.

Occasionally clayey sands, poorly graded sands, well graded sands will be exposed within the cut slopes. These types of soil have relatively high erosion potential.

Therefore, protective measures should be adopted to mitigate erosion potential of these soils if found necessary.

10.4 Dewatering

As described in the Section "Subsurface Soil Conditions", groundwater was not encountered at the project site during the field exploration performed in June 2010. Groundwater should unlikely be anticipated during excavations. However, the subsurface soil conditions consist of mainly clay with occasional pockets/lenses/layers of loose to medium dense sand, perched water table may be encountered if excavation is performed in winter or early spring. A perched water table (or perched aquifer) is an aquifer that occurs above the regional water table. This occurs when there is a relatively impermeable layer (aquitard) above the main water table/aquifer but below the ground surface.

If groundwater is encountered during excavation, groundwater may cause instability of excavation. Excessive water in the excavation may also result in difficult working conditions at the bottom of excavation causing subsequent delays in work and/or additional efforts during construction. Unstable excavation may cause slope failures, damage the shoring system, etc., causing excessive settlements of surrounding ground, damage to adjacent underground and above ground utilities and structures and excessive long-term differential settlements in the pipelines. Therefore, the Contractor should implement a dewatering system to mitigate these conditions.

All dewatering schemes proposed by the Contractor should be submitted to the Owner prior to implementation. However, developing and implementing an effective dewatering program should be the Contractor's responsibility. A properly designed and constructed dewatering operation is recommended irrespective of the construction method used. This may include, but not limited to, continuous inter-locking sheet piles, deep soil mix walls or other appropriate methods. The Contractor should install a dewatering system that will maintain the groundwater level below the bottom of the excavation at all times. In some areas that have conditions, which are difficult to effectively dry the bottom of the excavation, it may be necessary to maintain the groundwater at a deeper depth.

All dewatering systems should be properly designed to prevent pumping soil fines with the discharge water. The Contractor should sample and test the groundwater for soil fine content from the wells, as needed. If soil fines are being pumped, the Contractor should revise his dewatering operations; otherwise, it may result in failure of shoring, partial instability of excavation bottom resulting in intolerable settlement of foundations of existing structures and utilities and unsafe working conditions. The Contractor should provide discharge sampling locations for each pump. Prior to excavation, the Contractor should install observation wells at frequent locations along the alignment, to verify and demonstrate to the City of Mountain View that the required drawdown has been attained. Contractor is encouraged to perform their own investigation, test program, etc. prior to construction in order to satisfy their design requirements for an effective dewatering program.

10.5 Sand/Silt Pockets/Lenses/Layers

During our investigation, pockets/lenses/layers of silty sand/clayey sand and non-plastic silt were encountered at various depths in the soil borings. Surficial drying of these granular soils exposed along the excavation slopes may result in sloughing and caving. Adequate surface protection should be provided to the slope surface after its exposure from excessive drying and/or saturation during construction. To minimize sloughing and caving of these soils, they should be kept moist (but not saturated) by occasional light spraying of water during construction.

These pockets/lenses/layers of loose sands and non-plastic silt could not be accounted for in the slope stability analyses because of their variation in existence. These sand/silt pockets/lenses/layers may cause localized slope instability. A contingency plan and construction details will have to be developed so that during construction, the slope can be mitigated if these pockets/lenses/layers are encountered. PARIKH should be contacted to review these conditions in the field.

11.0 PLAN REVIEW

This report is prepared for the proposed “Permanente Creek Flood Improvement Project –

Flood Protection Improvement McKelvey Park". We recommend that final plans for the proposed project to be reviewed by PARIKH prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred. However, design-build elements should be reviewed only from overall compliance standpoint.

12.0 CONSTRUCTION OBSERVATION

To a degree, the performance of any structure is dependent upon construction procedures and quality control measures. Hence, geotechnical observation and testing of grading operations, and foundation excavations should be carried out by the Geotechnical Engineer. If the subsurface conditions different from those forming the basis of our recommendations are encountered, this office should be informed in order to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

13.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the subsurface conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the subsurface conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

REGISTERED PROFESSIONAL ENGINEER
Y. DAVID WANG
No. P.E. 52911
Exp. 12/31/14
CIVIL
STATE OF CALIFORNIA

Y. David Wang

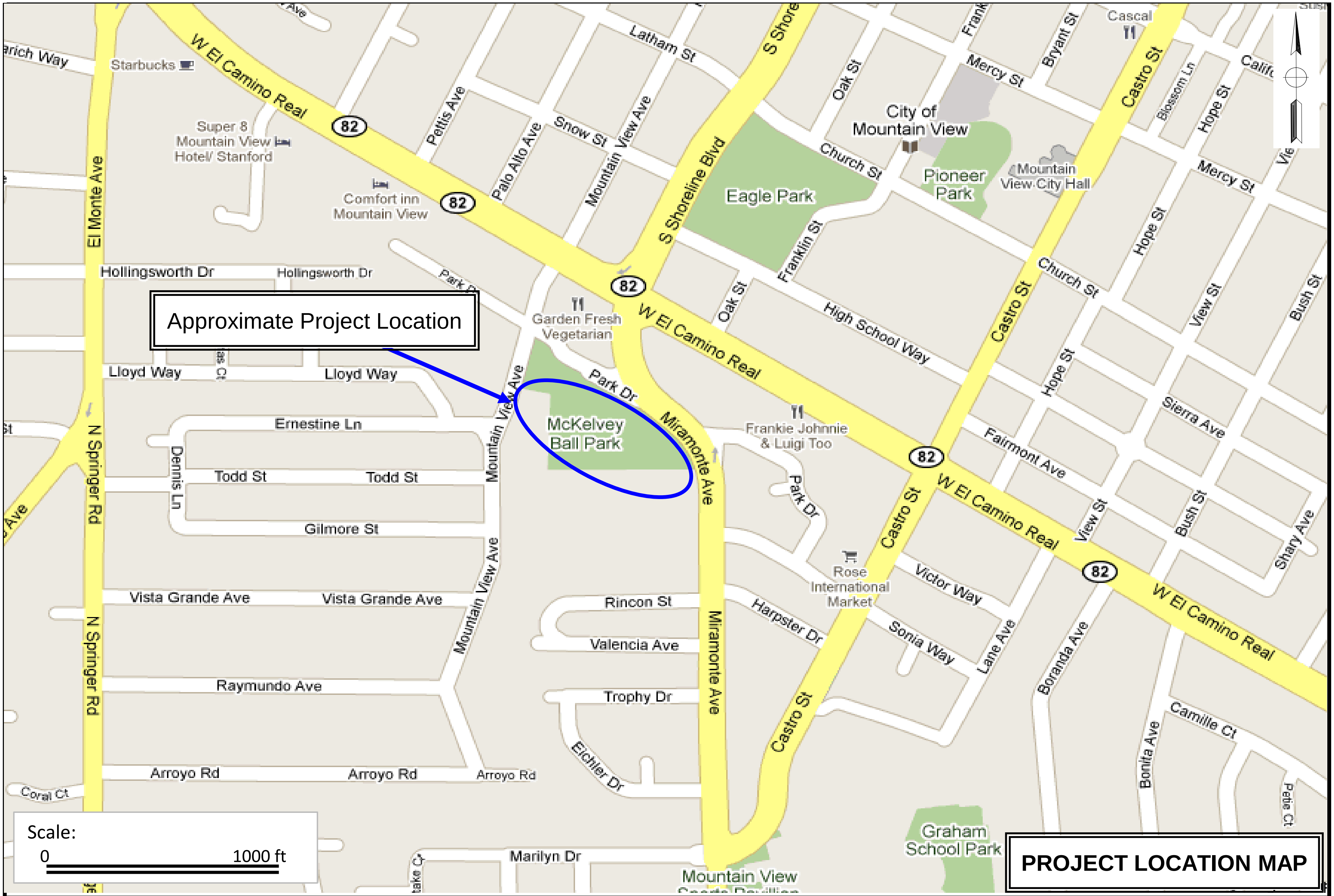
Alston Lam, P.E., G.E. 2605
Project Engineer

Y. David Wang, Ph.D., P.E. 52911
Project Manager

AL/GP: S:\Ongoing Projects\2010\210108\Permanente Creek Flood Improvement Project – McKelvey Park (9-2014).rpt

REFERENCES

1. “Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California”, a digital database, compiled by E.J. Helley, R.W. Graymer, G.A. Phelps, P.K. Showalter, and C.M. Wentworth, May 1994
2. California Guidelines for Structures Foundation Reports (Version 2.0) Updated December 2009.
3. Caltrans Department of Transportation, Caltrans ARS Online “dap3.dot.gov/ARS_Online/” (v2.3.06).



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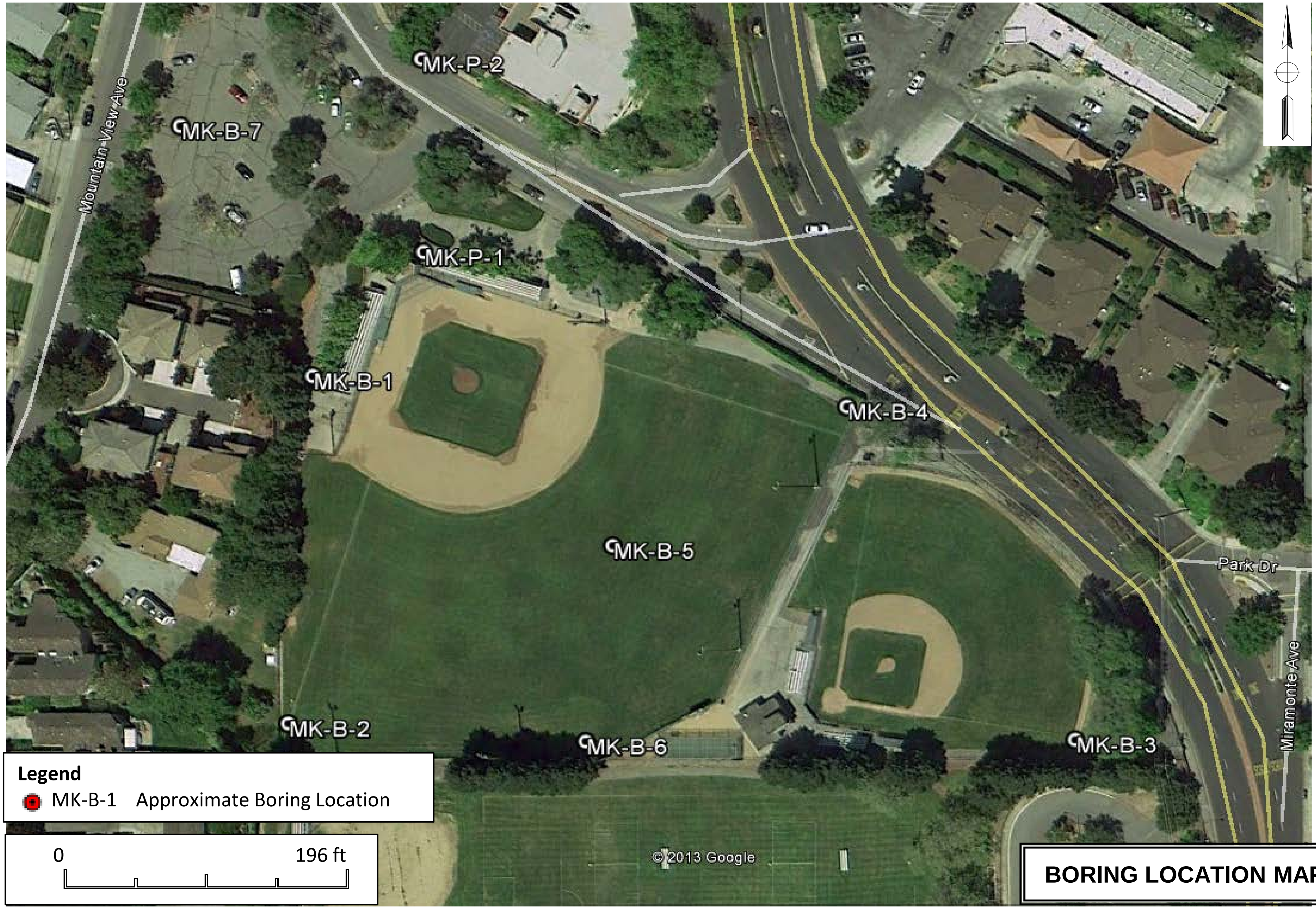
PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-010	**PLATE NO.: 1**



BORING LOCATION MAP

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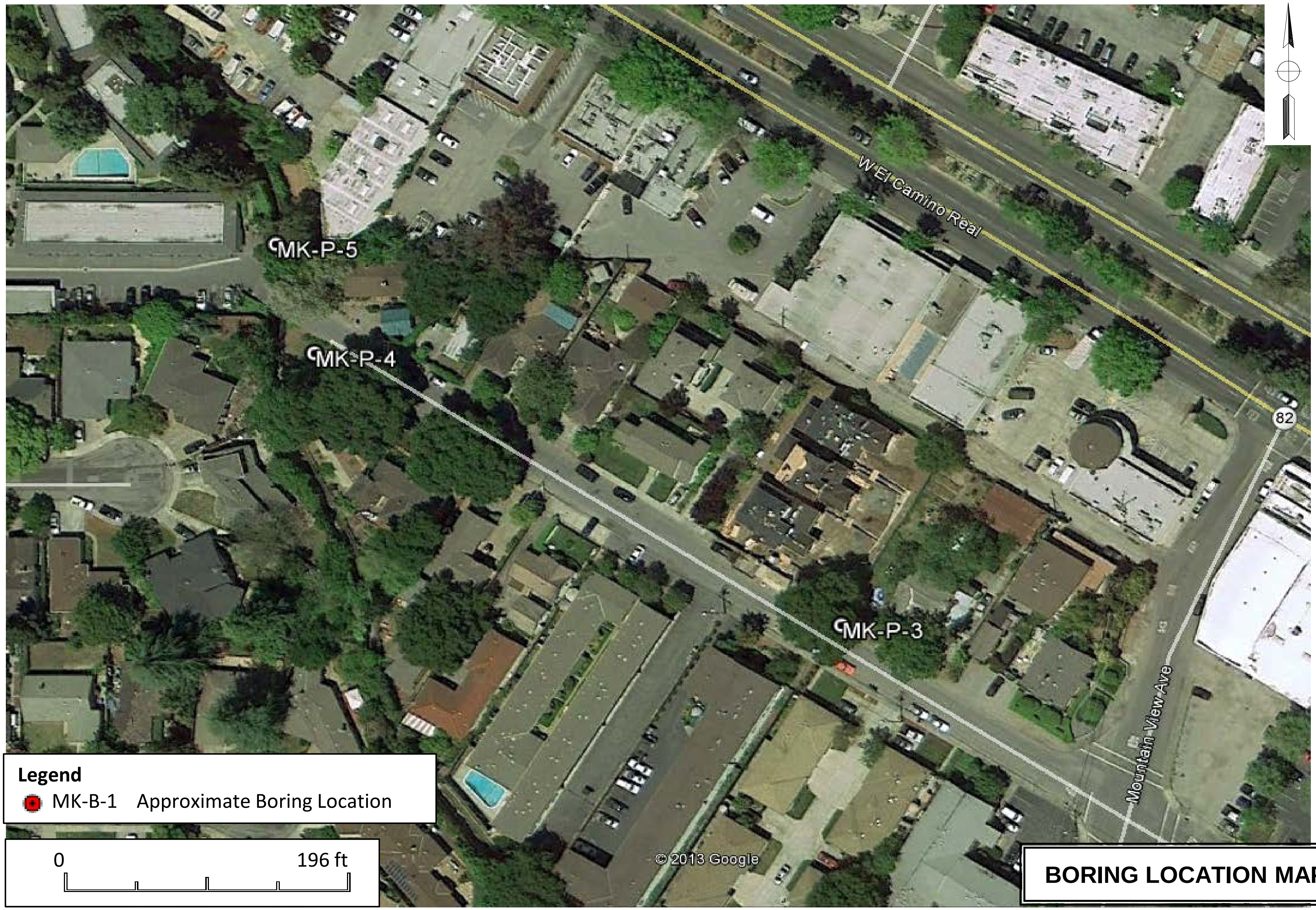
PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-010

PLATE NO.: 2A



BORING LOCATION MAP

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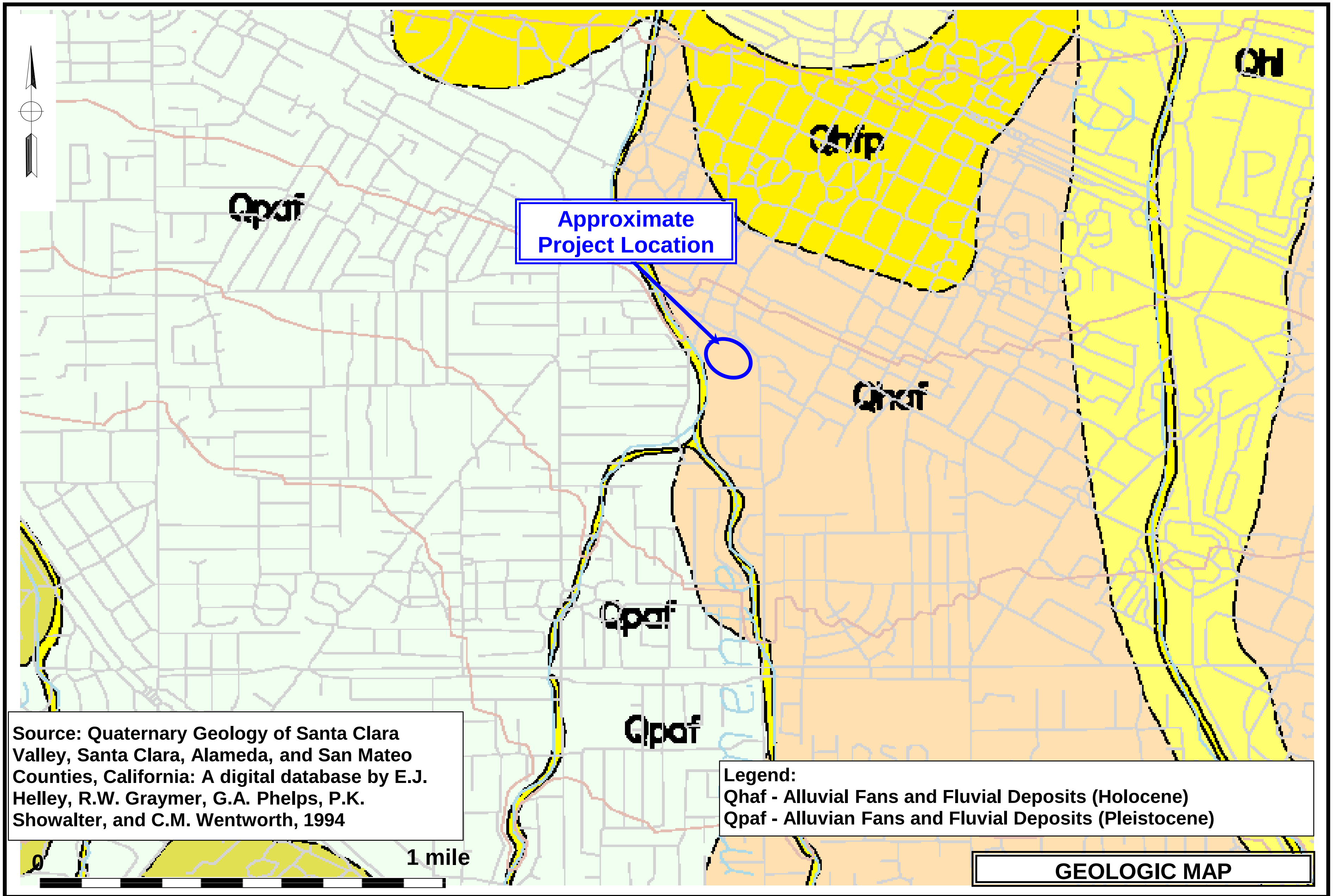
PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-010	**PLATE NO.: 2B**



Source: Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California: A digital database by E.J. Helley, R.W. Graymer, G.A. Phelps, P.K. Showalter, and C.M. Wentworth, 1994

Legend:
Qhaf - Alluvial Fans and Fluvial Deposits (Holocene)
Qpaf - Alluvian Fans and Fluvial Deposits (Pleistocene)

GEOLOGIC MAP

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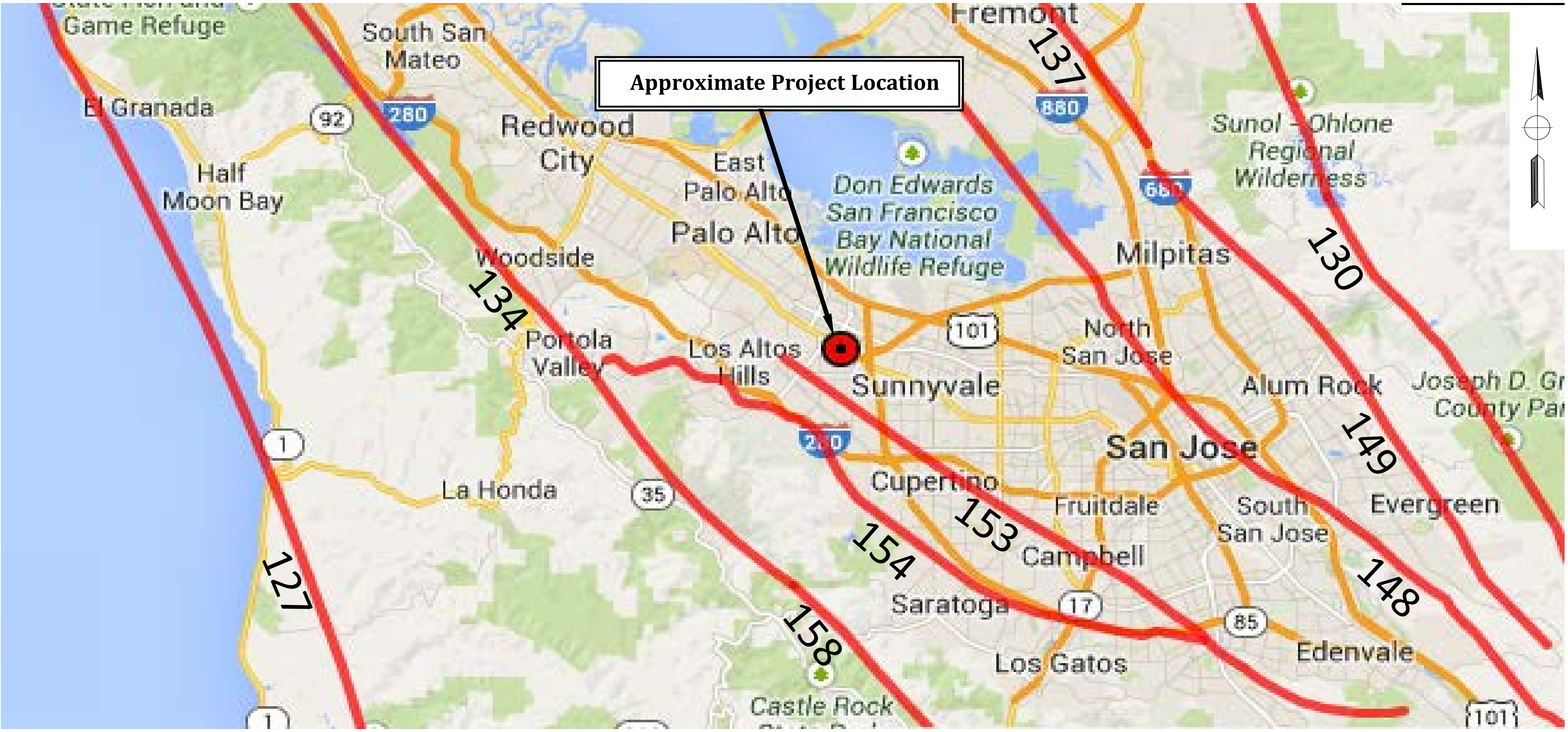
PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-010	PLATE NO.: 3



Source: Caltrans ARS Online Web Site
http://dap3.dot.ca.gov/ARS_Online/

Legend

153 - Cascade Fault (Mmax=6.7)
134 - San Andreas (Peninsula) 2011 CFM (Mmax=8.0)
158 - San Andreas (Santa Cruz Mts) 2011 CFM (Mmax=8.0)
154 - Monte Vista-Shannon (Mmax=6.4)
148 - Silver Creek (Mmax=6.9)
137 - Hayward (South) (Mmax=7.3)
127 - San Gregorio fault (San Gregorio section) (Mmax=7.4)
149 - Hayward (Southern Extension) (Mmax=6.7)
130 - Calaveras (No) 2011 CFM (Mmax=6.9)

Scale

0 10 miles

Fault Map

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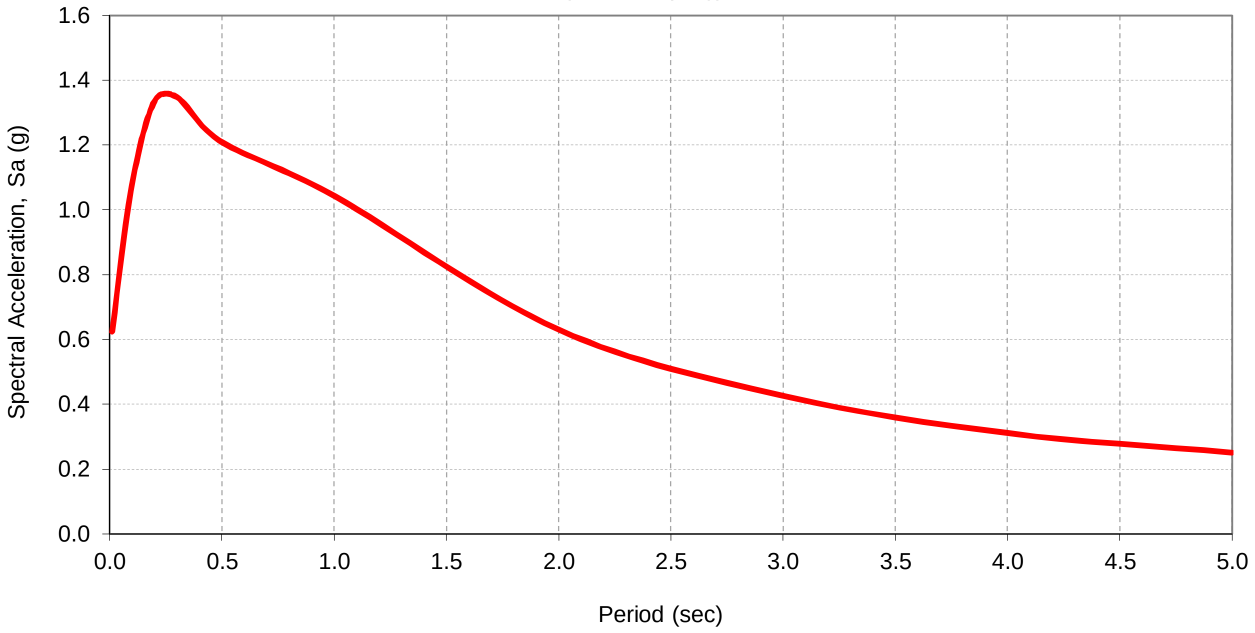
PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-010	PLATE NO.: 4

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

(5% Damping)



Site Information

Latitude:	37.3864
Longitude	-122.0889
V_{S30} (m/s) =	260
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	1.77

Governing Curve:

Caltrans Online Probabilistic ARS

Recommended Response Spectrum

Period (sec)	Caltrans Online Probabilistic Spectral Acceleration (g)	Adjusted for Near Fault Effect	Adjusted For Basin Effect	Final Adjusted Spectral Acceleration (g)
0.0	0.624	1	1	0.624
0.1	1.081	1	1	1.081
0.2	1.336	1	1	1.336
0.3	1.347	1	1	1.347
0.5	1.209	1	1	1.209
1.0	0.869	1.2	1	1.043
2.0	0.525	1.2	1	0.630
3.0	0.355	1.2	1	0.426
4.0	0.259	1.2	1	0.311
5.0	0.209	1.2	1	0.251

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)
2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)
3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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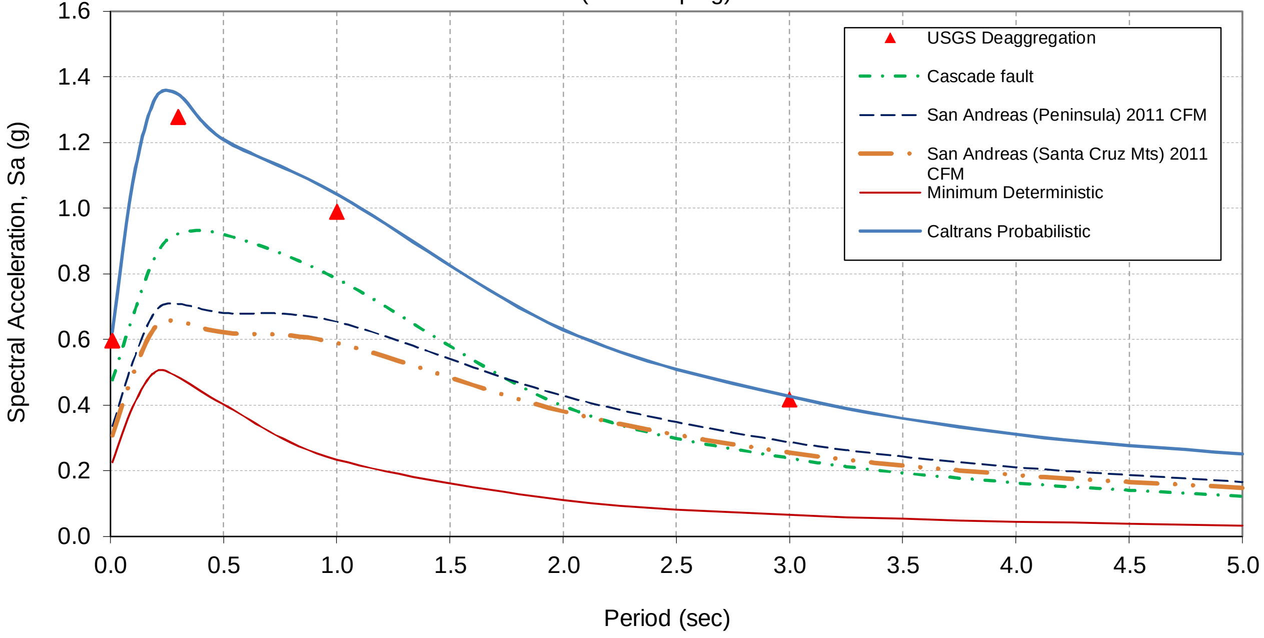
PERMENANTE CREEK FLOOD PROTECTION PROJECT
MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Project No.: 2010-108-010 | **Plate No.: 5A**

ACCELERATION RESPONSE SPECTRUM COMPARISON

(Deterministic & Probablistic Curves)
(5% Damping)



Period (sec)	Final Adjusted Spectral Accelerations (g)					
	Cascade fault	San Andreas (Peninsula) 2011 CFM	San Andreas (Santa Cruz Mts) 2011 CFM	Minimum Deterministic	Caltrans Probabilistic	USGS Deaggregation
0.0	0.477	0.336	0.308	0.226	0.624	0.597
0.1	0.672	0.532	0.494	0.396	1.081	
0.2	0.854	0.686	0.639	0.502	1.336	
0.3	0.922	0.708	0.655	0.485	1.347	1.277
0.5	0.920	0.681	0.621	0.402	1.209	
1.0	0.785	0.654	0.589	0.234	1.043	0.989
2.0	0.397	0.428	0.381	0.111	0.630	
3.0	0.238	0.287	0.255	0.066	0.426	0.417
4.0	0.163	0.211	0.187	0.045	0.311	
5.0	0.122	0.166	0.148	0.033	0.251	

Site Information

Latitude:	37.3864
Longitude	-122.0889
V_{S30} (m/s) =	260
$Z_{1.0}$ (m) =	N/A
$Z_{2.5}$ (km) =	N/A
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	1.77

Source:

1. Caltrans ARS Online tool (V.2, http://dap3.dot.ca.gov/ARS_Online/)

2. USGS Deaggregation 2008 beta (http://eqint.cr.usgs.gov/deaggint/2008/index.php)

3. Caltrans Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations, November 2012

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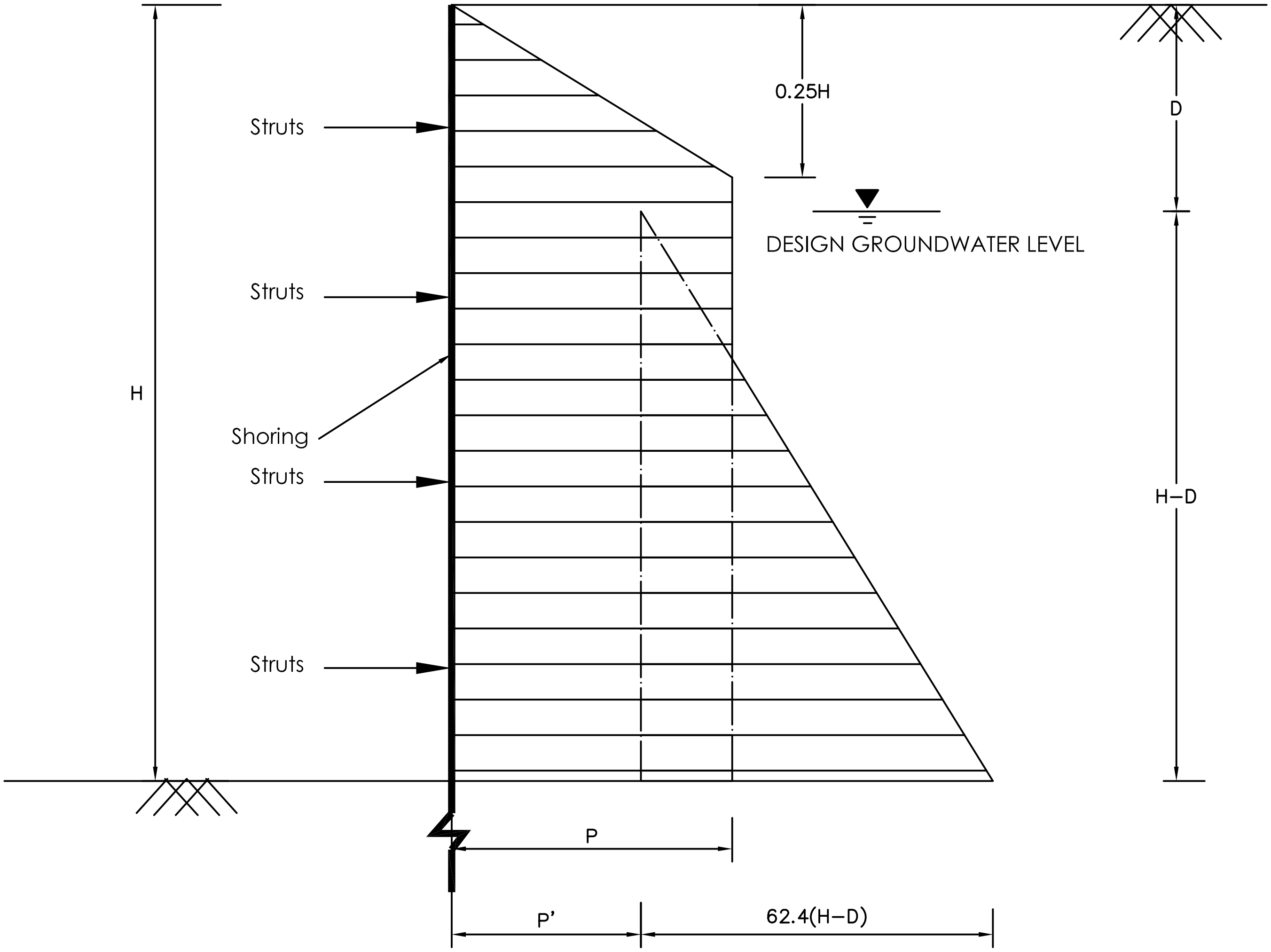
PERMENANTE CREEK FLOOD PROTECTION PROJECT
MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Project No.: 2010-108-010	**Plate No.: 5B**

SHORING LATERAL PRESSURE DIAGRAM

No Scale



D = Depth to groundwater table (ft)
H = Depth to bottom of trench (ft)
P = 50H (Pressure in psf)
P' = 27H (Pressure in psf)

NOTE:

1. All surchagre loads should be added to the horizontal loads at a rate of 40% of the total applied loads.
2. The pressure distribution is based on average soil conditions. The pressure should be re-evaluated where conditions are different.
3. The pressure distribution should be used for preliminary cost estimates only; the contractor should be responsible for his own design.

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENT McKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

JOB NO.: 2010-108-10

PLATE NO.: 6

APPENDIX A

APPENDIX A
FIELD EXPLORATION

All the soil borings were drilled with drill rig using 8-inch diameter hollow stem auger. The soil samples were obtained from the borings during drilling at various depths by driving a 2.5 inches Inside Diameter (I. D.) Modified California Sampler or a 1.375 inches I.D. Standard Penetration Sampler (ASTM Test Method No. 1586). The sampler was driven into the subsurface soils under the impact of a 140 pounds hammer having a free fall of 30 inches. The blow counts required to drive the sampler for the last 12 inches are presented on the Logs of Test Borings, Appendix A. (When correlating standard penetration data in similar soils, the blow counts for the Modified California sampler can be taken as roughly twice that for the Standard Penetration Test sampler in similar soils.). Pocket penetration tests were also performed on clay samples to evaluate their consistency. Upon completion of drilling, the boreholes were backfilled with cement grout.

The borings were drilled under the technical supervision of our engineers, who classified and continuously logged the soils encountered during drilling and supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were visually classified in the field according to the Unified Soil Classification System and then transported to our laboratory for further evaluation and testing.

The descriptions of the soils encountered and relevant boring information are presented on the Log of Borings in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs.

The descriptions and related information presented on these logs of test borings depict subsurface conditions only at the locations indicated on the plan and on the particular date noted on the logs. Because of the variability from place to place within soil/rock in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

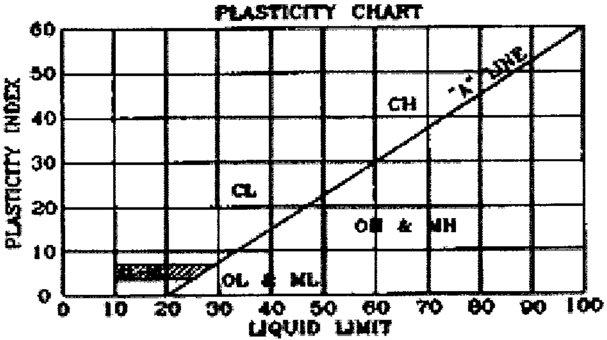
UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	ILLUSTRATIVE GROUP NAMES
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines	GW	Well graded gravel, Well graded gravel with sand
			GP	Poorly graded gravel, Poorly graded gravel with sand
		GRAVELS WITH FINES More than 12% fines	GM	Silty gravel, Silty gravel with sand
			GC	Clayey gravel, Clayey gravel with sand
	SANDS 50% or more of coarse fraction passing No. 4 sieve	CLEAN SANDS Less than 5% fines	SW	Well graded sand, Well graded sand with gravel
			SP	Poorly graded sand, Poorly graded sand with gravel
		SANDS WITH FINES More than 12% fines	SM	Silty sand, Silty sand with gravel
			SC	Clayey sand, Clayey sand with gravel
FINE-GRAINED SOILS 50% or more passing No. 200 sieve	SILTS AND CLAYS Liquid Limit less than 50%		ML	Silt, Sandy silt with gravel
			CL	Lean clay, Sandy lean clay with gravel
			OL	Organic clay, Sandy organic clay with gravel
	SILTS AND CLAYS Liquid Limit more than 50%		MH	Elastic silt, Sandy elastic silt with gravel
			CH	Fat clay, Sandy fat clay with gravel
			OH	Organic clay, Sandy organic clay with gravel
HIGHLY ORGANIC			PT	Peat, Highly organic silts

NOTE: 1. Coarse-grained soils receive dual symbols if: (a) their fines are CL-ML (e.g. SC-SM or GC-GM) or (b) they contain 5-12% fines (e.g. SW-SM, GP-GC, etc.). Fine-grained soils receive dual symbols if their limits plot in the hatched zone of the Plasticity Chart (CL-ML).

2. The table lists 30 out of a possible 110 Group Names, all of which are assigned to unique proportions of the constituent soils. Flow charts in ASTM D 2487-93 aid assignment of the Group Names

PLASTICITY CHART



GRAIN SIZE CLASSIFICATION

CLASSIFICATION	US STANDARD SIEVE SIZE
BOULDERS	Above 12"
COBBLES	12" to 3"
GRAVEL Coarse Fine	 3" to 3/4" 3/4" to No. 4
SAND Coarse Medium Fine	 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No.200
SILT & CLAY	Below No. 200

COHESIVE SOIL CONSISTENCY

CLASSIFICATION	UNCONFINED COMP. STRENGTH (psf)
Very Soft	< 500
Soft	500 - 1000
Medium(Firm)	1000 - 2000
Stiff	2000 - 4000
Very Stiff	4000 - 8000
Hard	> 8000

COHESIONLESS SOIL RELATIVE DENSITY

CLASSIFICATION	SPT BLOW COUNTS (Blows/ft)
Very Loose	< 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	> 50

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PERMENANTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CA

JOB NO.: 2010-108-010

PLATE NO.: A-1

Boring Location, Elevation & Date Drilled:	Drilling Method:	**BORING NUMBER**
; Elev. approx. 106.6 ft.; drilled on 6-1-10	8-inch dia. HOLLOW STEM AUGER B-53	**MK-B-1**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	AC	4" AC over 4" AB (estimate only)	
						CL	Lean CLAY with SAND and GRAVEL (CL); stiff; dark brown; dry to moist.	
MC-1	88.4	10.4	21					pp=3.5tsf
MC-2	98.2	11.1	11	2.4	5		-firm	UC; (LL=31, PI=14)
MC-3	112.3	15.3	40		10		-very stiff; brown with dark and yellowish brown mottling, moist.	pp=>4.0tsf
						SC	CLAYEY SAND (SC); medium dense; brown; moist; fine to medium SAND.	
MC-4	113.2	10.4	33		15			pp=>4.0tsf
						SW SM	Well-graded SAND with SILT and GRAVEL (SW-SM); dense; grayish brown, dry to moist; weak cementation.	
MC-5	-	3.9	71		20			(+#4=29%, -#200=6%)
MC-6	-	6.6	50/6"		25		-very dense; fine to medium SAND.	
						SM	SANDY SILT/SILTY SAND (ML/SM); medium dense; brown; moist.	
MC-7	-	5.6	46		30			

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:

A-2A

Boring Location, Elevation & Date Drilled: ; Elev. approx. 106.6 ft.; drilled on 6-1-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-1**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.
					35	
					40	
					45	
					50	
					55	
					60	

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate: A-2B

LB_NEW_LOGO 210108.10.03.MP.GPJ 4-23-14

Boring Location, Elevation & Date Drilled: ; Elev. approx. 108.8 ft.; drilled on 6-2-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-2 (W)**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	Remarks
					0	CL	Lean CLAY (CL); firm; dark grayish brown; moist.	
MC-1	99.9	14.5	12					pp=>4.0tsf
MC-2	106.4	17.0	16		5		-stiff; dark brown with white mottling.	pp=>4.0tsf
MC-3	106.8	18.3	26	4.1	10		-very stiff.	UC, pp=2.3tsf
						CL	Lean CLAY with GRAVEL (CL); very stiff; brown; moist; fine SAND.	
MC-4	-	11.6	32		15			
MC-5	-	15.0	38		20		-brown with white mottling.	
MC-6	91.8	24.5	38		25		-very stiff, yellowish brown.	
						ML	SILT with SAND (ML); very stiff; brown; moist.	
MC-7	99.2	21.6	24	0.9	30			UC

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010
Job No.: 2010-108-010

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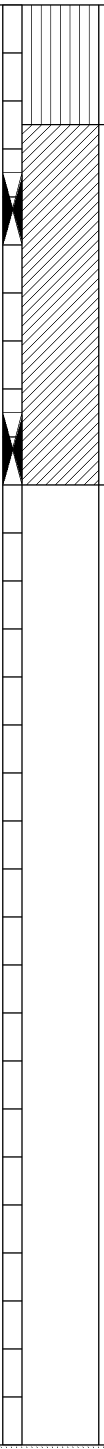
Plate: A-3A

LB_NEW_LOGO 210108.10.03.MP.GPJ 4-23-14

Boring Location, Elevation & Date Drilled: ; Elev. approx. 108.8 ft.; drilled on 6-2-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-2 (W)**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description
					30		
						CL	Lean CLAY with SAND and GRAVEL (CL); very stiff, brownish gray; moist.
MC-8	106.4	22.6	44		35		
MC-9	-	16.8	26		40		
							Bottom of the hole at 40 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion. Monitoring well was installed with 2 inches PVC pipe (0.01 inch slots); 20 feet of screen topped with 20.0 feet blank; blanket of #3 sand from 40.0 feet to 18 feet (below ground); bentonite from 18 feet to 16 feet (below ground); cement grout from 16 feet to 6 inches (below ground); 8 inches well box installed.
					45		
					50		
					55		
					60		

LOG OF BORING

PARIKH
Practicing in the Geosciences

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

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Plate:
A-3B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 107.2 ft.; drilled on 6-1-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-3**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	CL	Lean CLAY with SAND (CL); stiff; dark brown; moist.	
MC-1	112.4	18.0	18					pp=3.8tsf
						SC	CLAYEY SAND with GRAVEL (SC); very loose; dark brown; moist; fine to coarse SAND; fine GRAVEL up to 1/2"; weak cementation.	
MC-2	112.9	9.3	6		5			(+#4=20%, -#200=23%)
						ML	SILT with SAND (ML); hard; dark brown; moist; angular GRAVEL up to 1.5".	
MC-3	-	4.3	54		10			
						CL	Lean CLAY with SAND (CL); firm to stiff; brown; moist.	
MC-4	111.2	16.8	17		15			(LL=35, PI=16), pp=>4.0tsf
						CL	SANDY lean CLAY (CL); stiff; brown; moist.	
MC-5	99.4	16.2	19		20			
						CL	Lean CLAY (CL); very sitff; tanish brown with yellowish brown mottling.	
MC-6	92.7	29.5	25	2.7	25			UC, pp=1.5tsf
MC-7	111.8	19.1	21		30		-stiff; greenish brown; GRAVEL up to 1/2".	pp=3.5tsf

LOG OF BORING

PARIKH
Practicing in the Geosciences

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

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Plate:
A-4A

Boring Location, Elevation & Date Drilled: ; Elev. approx. 107.2 ft.; drilled on 6-1-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-3**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.
					35	
					40	
					45	
					50	
					55	
					60	

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

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Plate:

A-4B

Boring Location, Elevation & Date Drilled:
; Elev. approx. 105.8 ft.; drilled on 6-1-10

Drilling Method:
8-inch dia. HOLLOW STEM AUGER
B-53

BORING NUMBER

MK-B-4

Sampling Method:
2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.

Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	CL	Lean CLAY (CL); firm; brown; moist; trace fine SAND; vegetation.	
MC-1	108.3	19.3	11					pp=1.5tsf
MC-2	108.4	18.6	13					pp=2.0tsf
					5			
MC-3	110.7	16.4	13	1.1				UC
							-fine SAND; trace coarse GRAVEL.	
					10			
						SC SM	SILTY, CLAYEY SAND (SC-SM); loose; brown; moist; fine to medium SAND; fine GRAVEL.	
MC-4	140.0	13.7	10					(+#4=12%, -#200=27%)
					15			
						ML	SILT with SAND and GRAVEL (ML); very stiff; brown; moist; coarse SAND.	
MC-5	-	4.8	41					
					20	CL	Lean CLAY (CL); very stiff; light tanish brown with dark brown mottling; moist; few coarse GRAVEL.	
MC-6	101.0	25.4	31					pp=>4.0tsf
					25			
						SM	SILTY SAND with GRAVEL (SM); medium dense; brown; moist; fine to medium SAND; coarse GRAVEL; weak cementation.	
MC-7	108.3	12.2	27		30			

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

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Plate:

A-5A

LB_NEW_LOGO 210108.10.03.MP.GPJ 4-23-14

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
; Elev. approx. 105.8 ft.; drilled on 6-1-10	8-inch dia. HOLLOW STEM AUGER B-53	**MK-B-4**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.
					35	
					40	
					45	
					50	
					55	
					60	

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

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Plate:

A-5B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 107.6 ft.; drilled on 6-2-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-5**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	Remarks
					0	CL	Lean CLAY (CL); soft to firm; dark grayish brown; wet.	
MC-1	101.9	22.8	7	0.5				UC, pp=0.3tsf
MC-2	110.0	19.1	9		5		-firm; brown; very moist.	pp=1.5tsf
MC-3	107.6	21.0	10		10		-dark brown.	CN; (LL=41; PI=19)
						SC	CLAYEY SAND with GRAVEL (SC); loose; dark brown; very moist.	
MC-4	-	13.6	10		15			CR
MC-5	120.7	14.4	13		20		-brown	
						CL	SANDY lean CLAY (CL); stiff; brownish gray; very moist.	
MC-6	97.1	25.7	27		25			pp=3.5tsf
						CL	Lean CLAY (CL); stiff; grayish brown; very moist.	
MC-7	101.4	25.9	24		30		-light greenish brown with dark brown mottling.	pp=3.0tsf

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010
Job No.: 2010-108-010

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Plate:
A-6A

Boring Location, Elevation & Date Drilled: ; Elev. approx. 107.6 ft.; drilled on 6-2-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-5**

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.	
					35		
					40		
					45		
					50		
					55		
					60		

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010
Job No.: 2010-108-010

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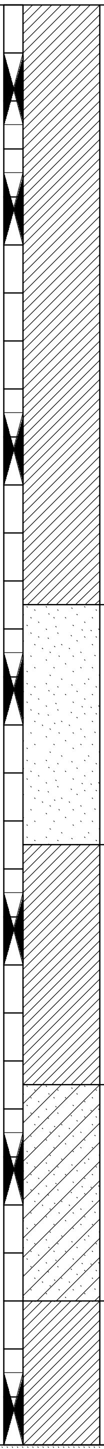
Plate:
A-6B

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
; Elev. approx. 108.3 ft.; drilled on 6-2-10	8-inch dia. HOLLOW STEM AUGER B-53	**MK-B-6**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					0	CL	Lean CLAY (CL);stiff; dark brown; moist.	
MC-1	108.9	18.1	23					pp=>4.0tsf
MC-2	105.8	18.0	22	3.3	5			UC, pp=>4.0tsf
MC-3	-	-	15		10			
						SP	Poorly graded SAND with GRAVEL (SP); medium dense; brown; moist; coarse SAND; fine GRAVEL; weak cementation.	
MC-4	-	-	27		15			
						CL	SANDY lean CLAY (CL); very stiff; tanish brown; moist.	
MC-5	113.1	18.7	27		20			pp=3.8tsf
						SC	CLAYEY SAND (SC); medium dense; yellowish brown; coarse SAND; moderate cementation.	
MC-6	126.0	9.4	54		25			
						CL	SANDY lean CLAY (CL); very stiff; light greenish brown; moist.	
MC-7	106.5	22.1	33		30			pp=3.5tsf

LOG OF BORING

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010 Job No.: 2010-108-010

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Plate: A-7A

Boring Location, Elevation & Date Drilled: ; Elev. approx. 108.3 ft.; drilled on 6-2-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-6**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.
					35	
					40	
					45	
					50	
					55	
					60	

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

Job No.: 2010-108-010

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Plate:

A-7B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 105.8 ft.; drilled on 6-1-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-B-7**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.		Description	
					0	AC	4" AC over 4" AB	
						CL	Lean CLAY with SAND and GRAVEL (CL); stiff; dark brown; moist.	
MC-1	107.8	10.9	23					pp=>4.0tsf
MC-2	98.9	9.6	15		5		-stiff; brown to dark brown; fine SAND.	pp=>4.0tsf
MC-3	111.1	12.0	28		10		-very stiff; dark brown with white mottling; dry to moist.	pp=>4.0tsf
MC-4	-	10.4	20		15		-stiff; dark brown; moist; coarse SAND; fine GRAVEL; moderate cementation.	pp=>4.0tsf
MC-5	116.3	15.5	22		20		-tanish brown with black mottling.	pp=>4.0tsf
						SC	CLAYEY SAND with GRAVEL (SC); medium dense; brown; moist; fine to medium SAND.	
MC-6	119.0	8.8	36		25			pp=>4.0tsf
						CL	SANDY lean CLAY (CL); stiff; brown with black mottling; moist; fine SAND.	
MC-7	99.3	27.0	22		30			(+#4=0%, -#200=65%), pp=1.8tsf

LOG OF BORING

PARIKH
Practicing in the Geosciences

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010 Job No.: 2010-108-010

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Plate: A-8A

Boring Location, Elevation & Date Drilled:
; Elev. approx. 105.8 ft.; drilled on 6-1-10

Drilling Method:
8-inch dia. HOLLOW STEM AUGER
B-53

BORING NUMBER

MK-B-7

Sampling Method:
2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.

Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft) Soil Graph & U.S.C.S.	
					30	Bottom of the hole at 30 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.
					35	
					40	
					45	
					50	
					55	
					60	

LB_NEW_LOGO 210108.10.03.MP.GPJ 4-23-14

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MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

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Plate:

A-8B

Boring Location, Elevation & Date Drilled:
; Elev. approx. 105.6 ft.; drilled on 6-1-10

Drilling Method:
8-inch dia. HOLLOW STEM AUGER
B-53

BORING NUMBER

MK-P-1

Sampling Method:
2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.

Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	AC	4" AC over 4" AB	
						CL	SANDY lean CLAY with GRAVEL (CL); stiff to very stiff; dark brown; moist.	
MC-1	112.4	15.5	18					pp=>4.0tsf
						ML	SANDY SILT with GRAVEL (ML); firm; dark brown; moist; fine GRAVEL.	
MC-2	106.0	12.3	10		5			-#200=56%, pp=>4.0tsf
						CL	Lean CLAY with GRAVEL (CL); very stiff; dark brown; moist.	
MC-3	114.7	16.4	26		10			pp=>4.0tsf
						SC	CLAYEY SAND with GRAVEL (SC); loose; dark brown; moist.	
MC-4	113.0	12.4	16		15			+#4=7%, -#200=32%
MC-5	118.5	12.6	26		20		-medium dense; olive brown; medium SAND; fine GRAVEL.	
MC-6	-	7.8	38		25		-coarse SAND; strong cementation.	
						CL	Lean CLAY with GRAVEL (CL); very stiff; tanish brown with orange brown mottling; moist.	
MC-7	100.3	26.8	34		30			pp=2.5tsf

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PARIKH
Practicing in the Geosciences

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010

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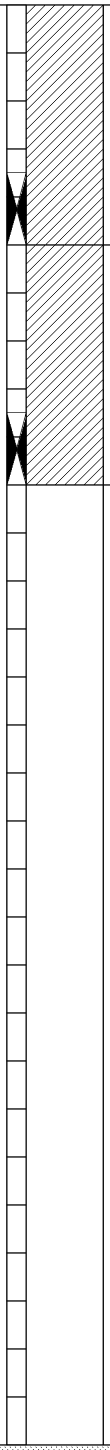
Plate:
A-9A

Boring Location, Elevation & Date Drilled: ; Elev. approx. 105.6 ft.; drilled on 6-1-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-53	**BORING NUMBER** **MK-P-1**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					30		Lean CLAY with GRAVEL (CL)	
MC-8	92.7	23.9	15				-stiffl; tanish brown; fine SAND.	pp=1.0tsf
					35	CL	SANDY lean CLAY (CL); very stiff; tanish brown; moist.	
MC-9	116.3	17.8	36					
					40		Bottom of the hole at 40 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.	
					45			
					50			
					55			
					60			

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010
Job No.: 2010-108-010

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Plate:
A-9B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 103.8 ft.; drilled on 6-8-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-61	**BORING NUMBER** **MK-P-2**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	AC	4" AC over 8" AB	
						CL	Lean CLAY with GRAVEL (CL); stiff; dark brown; moist.	
MC-1	97.1	10.9	15					
MC-2	-	5.4	9		5		-firm; moist to dry; fine SAND	
						CL	Lean CLAY with SAND and GRAVEL (CL); stiff; dark brown with white mottling; moist.	
MC-3	109.4	13.1	23		10			CR
MC-4	110.5	14.7	10	1.3	15		-firm; brown.	UC; (LL=29, PI=11)
						ML	SILT with SAND and GRAVEL (ML); very stiff; grayish brown; moist; medium SAND; fine GRAVEL.	
MC-5	100.1	6.2	30		20			
						CL	Lean CLAY (CL); very stiff; dark brown; moist; trace medium SAND.	
MC-6	101.7	24.4	28		25			pp=1.5tsf
						ML	SILT with SAND and GRAVEL (ML); hard; olive brown; moist; few fine to medium SAND.	
MC-7	-	5.9	54		30			

LOG OF BORING

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Practicing in the Geosciences

PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

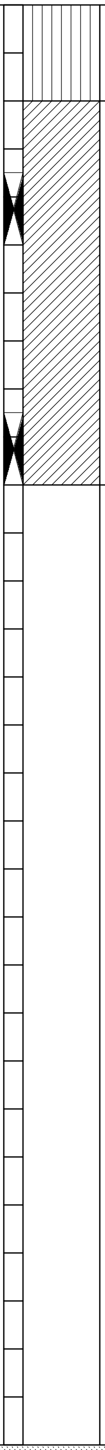
Date: 07/09/2010

Job No.: 2010-108-010

This log is part of the report prepared by Parikh Consultants, Inc. for the named project and should be read together with that report for complete interpretation. This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Plate:
A-10A

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
; Elev. approx. 103.8 ft.; drilled on 6-8-10	8-inch dia. HOLLOW STEM AUGER B-61	**MK-P-2**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					30		SILT with SAND and GRAVEL (ML)	
						CL	Lean CLAY with GRAVEL (CL); very stiff; light tanish brown with dark brown mottling; very moist; trace medium to coarse SAND;	
MC-8	98.7	26.9	30		35			
MC-9	102.4	23.6	24	2.3	40		Bottom of the hole at 40 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.	UC
					45			
					50			
					55			
					60			

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FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
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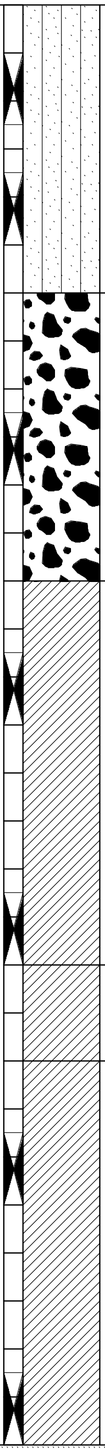
Plate:

A-10B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 104.8 ft.; drilled on 6-8-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-61	**BORING NUMBER** **MK-P-3**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					0	SM	SILTY SAND with GRAVEL (SM); very loose; dark grayish brown; moist	
MC-1	107.9	8.0	5					
MC-2	-	4.9	17		5		-loose; coarse GRAVEL.	
						GP	Poorly graded GRAVEL (GP); medium dense; brownish gray; dry to moist; angular; fine to coarse GRAVEL	
MC-3	-	1.7	53		10			
						CL	SANDY lean CLAY (CL); very stiff; dark brown; moist; fine to medium SAND.	
MC-4	-	11.4	29		15			pp=>4.0tsf
MC-5	112.1	15.2	28		20		-yellowish brown with dark brown mottling	pp=>4.0tsf
						CL	Lean CLAY with SAND (CL); very stiff; yellowish brown with dark brown mottling; moist.	
						CL	SANDY lean CLAY (CL); very stiff; light brown; moist.	
MC-6	97.1	25.7	24		25			pp=>4.0tsf
MC-7	106.6	19.2	29		30		-yellowish brown; moist	pp=>4.0tsf

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010 Job No.: 2010-108-010

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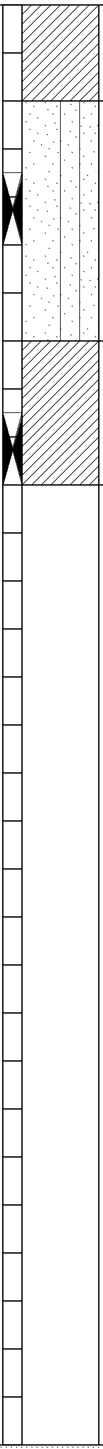
Plate: A-11A

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
; Elev. approx. 104.8 ft.; drilled on 6-8-10	8-inch dia. HOLLOW STEM AUGER B-61	**MK-P-3**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2



Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					30		SANDY lean CLAY (CL)	
						SP SM	Poorly graded SAND with SILT (SP-SM); medium dense; dark brown; moist; fine SAND.	
MC-8	103.5	9.7	36		35			+#4=1%, -#200=9%
						CL	Lean CLAY (CL); frim to stiff; light tanish brown with brown mottling; moist.	
MC-9	106.3	24.2	22		40			pp=2.8tsf
							Bottom of the hole at 40 feet. No groundwater was encountered during drilling. Drill hole was grouted upon completion.	
					45			
					50			
					55			
					60			

LOG OF BORING

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PERMANENTE CREEK FLOOD PROTECTION PROJECT
FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

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Plate:
A-11B

Boring Location, Elevation & Date Drilled: ; Elev. approx. 102.9 ft.; drilled on 6-8-10	Drilling Method: 8-inch dia. HOLLOW STEM AUGER B-61	**BORING NUMBER** **MK-P-4**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 1 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	Soil Graph & U.S.C.S.	Description	
					0	CL	SANDY lean CLAY with GRAVEL (CL); stiff; dark brown; dry to moist.	
MC-1	101.5	11.4	15					pp=3.3tsf
MC-2	91.8	14.7	9		5		-firm; moist; trace coarse GRAVEL.	pp=1.5tsf
						SP	Poorly GRADED SAND with GRAVEL (SP); medium dense; brown; moist; medium to coarse SAND; weak cementation.	
MC-3	108.2	4.2	22		10			-#200=3%
MC-4	-	-	33		15			
						CL	Lean CLAY (CL); very stiff; yellowish brown; moist.	
MC-5	108.9	19.5	26		20			pp=2.8tsf
						CL	Lean CLAY with GRAVEL (CL); stiff; brown with orangish brown mottling; moist.	
MC-6	114.4	13.2	23		25			
						ML	SILT (ML); very stiff; yellowish brown; moist.	
MC-7	101.9	24.0	32	1.2	30			UC; (LL=31, PI=8)

LOG OF BORING

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FLOOD PROTECTION IMPROVEMENTS - MCKELVEY PARK
MOUNTAIN VIEW, CALIFORNIA

Date: 07/09/2010
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Plate:
A-12A

Boring Location, Elevation & Date Drilled:	Drilling Method:	BORING NUMBER
; Elev. approx. 102.9 ft.; drilled on 6-8-10	8-inch dia. HOLLOW STEM AUGER B-61	**MK-P-4**
	Sampling Method: 2 1/2 " I.D. Mod. Calif. (MC), 140 lb hammer, 30 inch drop.	Sheet 2 of 2

Sample Type & No.	Dry Density (pcf)	Water Content (%)	Blows Per Foot	Compress. Strength (tsf)	Depth (ft)	U.S.C.S.	Description	
					30		SILT (ML)	
MC-8	112.7	18.4	24	2.8	35		-stiff; yellowish gray.	UC, pp=1.3tsf
						SW SM	Well-graded SAND with SILT and GRAVEL (SW-SM); medium dense; gray; wet; medium to coarse SAND; weak cementation.	
MC-9	109.9	12.4	46		▽ 40			
							Bottom of the hole at 40 feet. Groundwater was encountered @ 39.5 feet during drilling. Drill hole was grouted upon completion.	
					45			
					50			
					55			
					60			

LOG OF BORING

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Plate:
A-12B

APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented in “Log of Boring”, Appendix A.

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with California Test Method T-226. This information was used to classify and correlate the soils. The results are presented at the appropriate depths In "Log of Boring", Appendix A.

Atterberg Limits

The Atterberg Limits were determined for selected samples of the fine-grained materials. These results were used to classify the soils, as well as to obtain an indication of the effective strength characteristics and expansion potential with variations in moisture content. The Atterberg Limits were determined in general accordance with California Test Method T-204. The results of these tests are presented on Plate B-2,” Plasticity Chart”.

Grain Size Classification

Grain size classification tests (California Test Method T-202) were performed on selected samples of granular soil to aid in the classification. The results are presented on Plates B-3A and B-3B, “Grain Size Distribution Curves”.

Unconfined Compression Tests

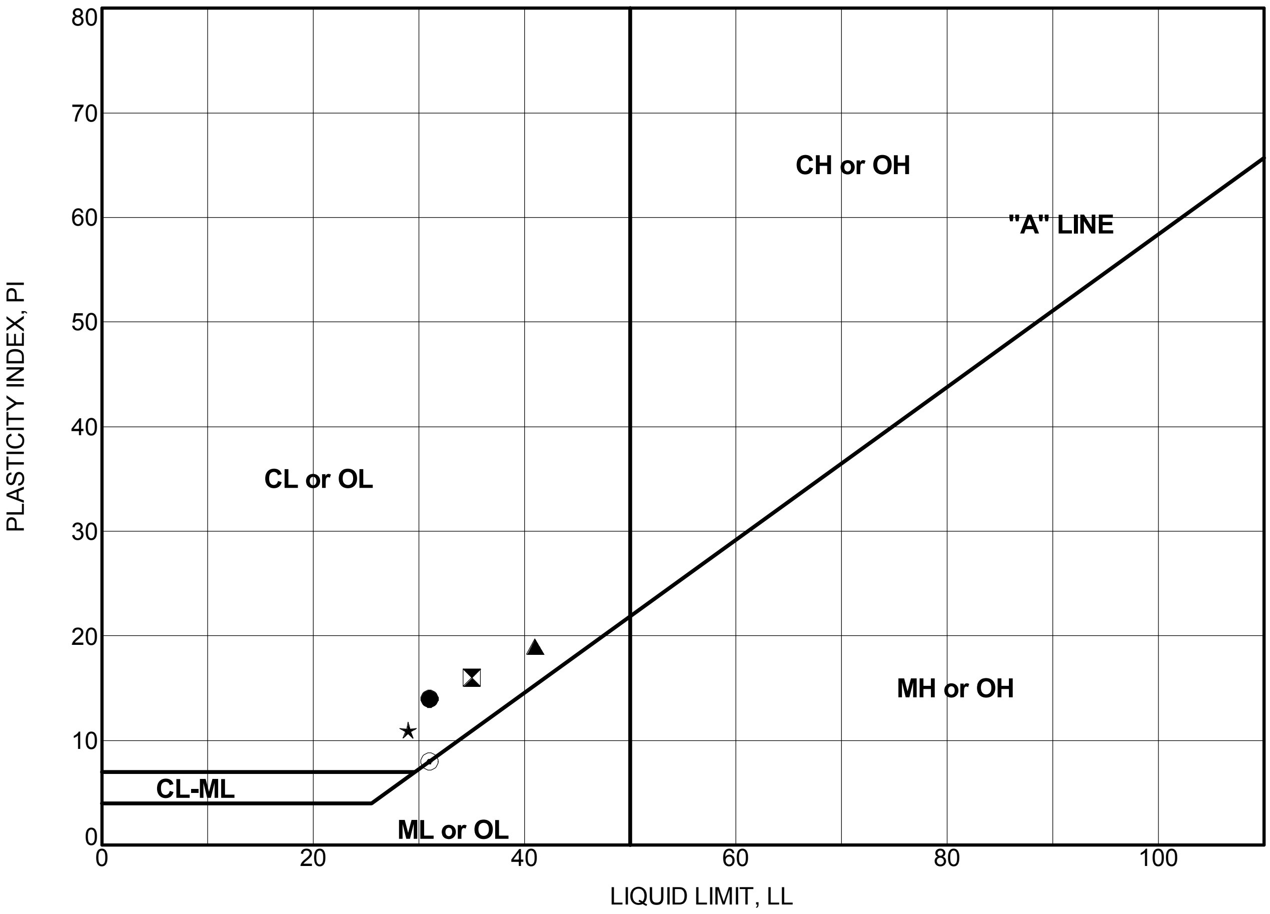
Strength tests were performed on selected undisturbed samples using unconfined compression machine. Unconfined compression tests were performed in general accordance with California Test Method T-221. The results are presented on Plates B-4A through B-4K.

Consolidation Tests

A consolidation test (ASTM Test Method D-2435) was performed on a selected undisturbed sample. The test results are presented on Plates B-5A and B-5B.

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistively tests were performed according to California Test Method T-643. The tests were performed by Sunland Analytical. The test results are presented on Plates B-6A and B-6B.



PLASTICITY CHART

Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
MK-B-1	MC-2	4.5	●	11	31	17	14	Lean CLAY with SAND and GRAVEL (CL)
MK-B-3	MC-4	14.5	⧗	17	35	19	16	Lean CLAY with SAND (CL)
MK-B-5	MC-3	9.5	▲	21	41	22	19	Lean CLAY (CL)
MK-P-2	MC-4	14.5	★	15	29	18	11	Lean CLAY with SAND and GRAVEL (CL)
MK-P-4	MC-7	29.5	⊙	24	31	23	8	SILT (ML)

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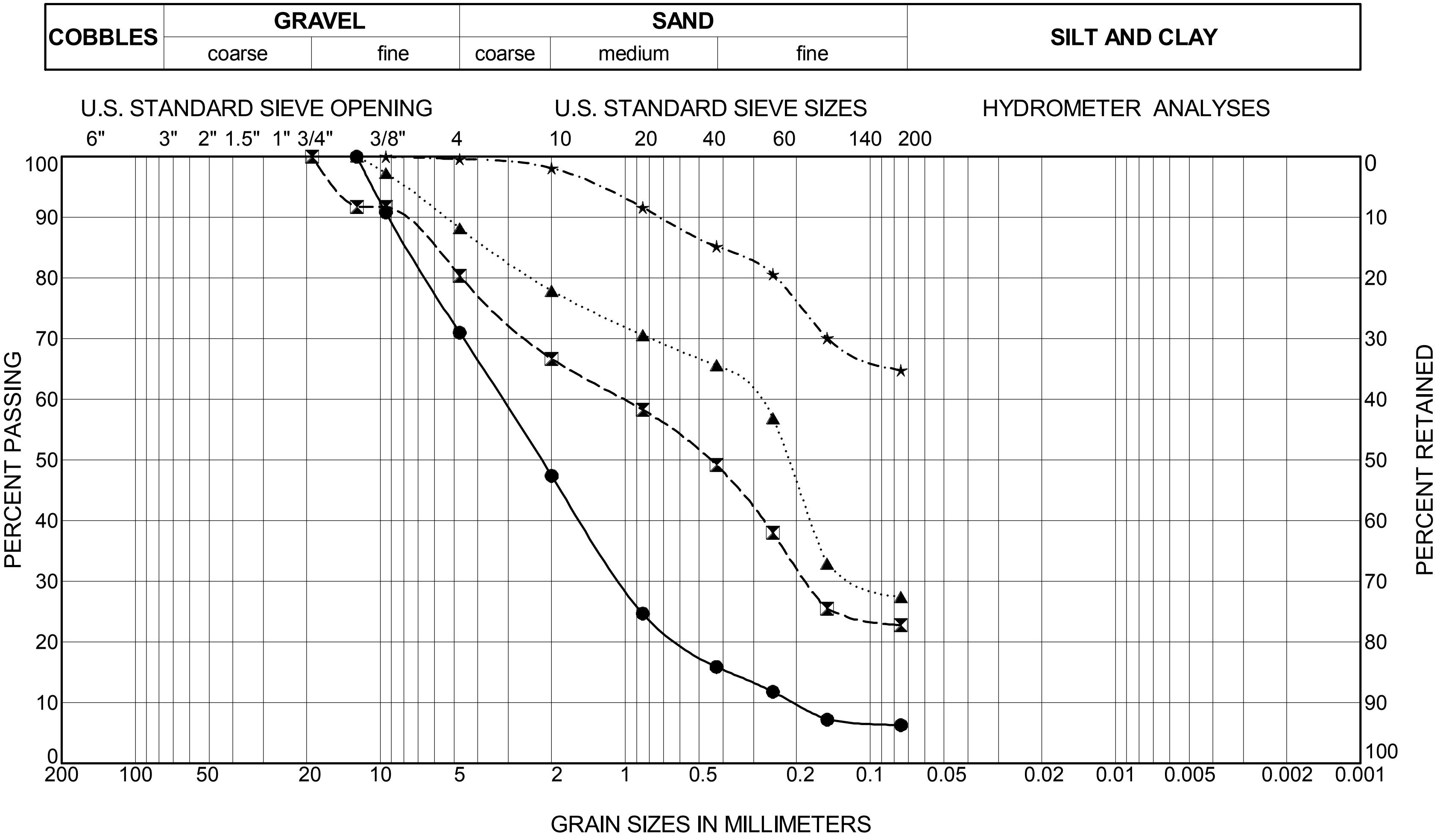
PLATE NO: B-2

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
MK-B-1	MC-5	19.5	●			Well-graded SAND with SILT and GRAVEL (SW-SM)
MK-B-3	MC-2	4.5	⧗			CLAYEY SAND with GRAVEL (SC)
MK-B-4	MC-4	13.0	▲			SILTY, CLAYEY SAND (SC-SM)
MK-B-7	MC-7	29.5	★			SANDY lean CLAY (CL)

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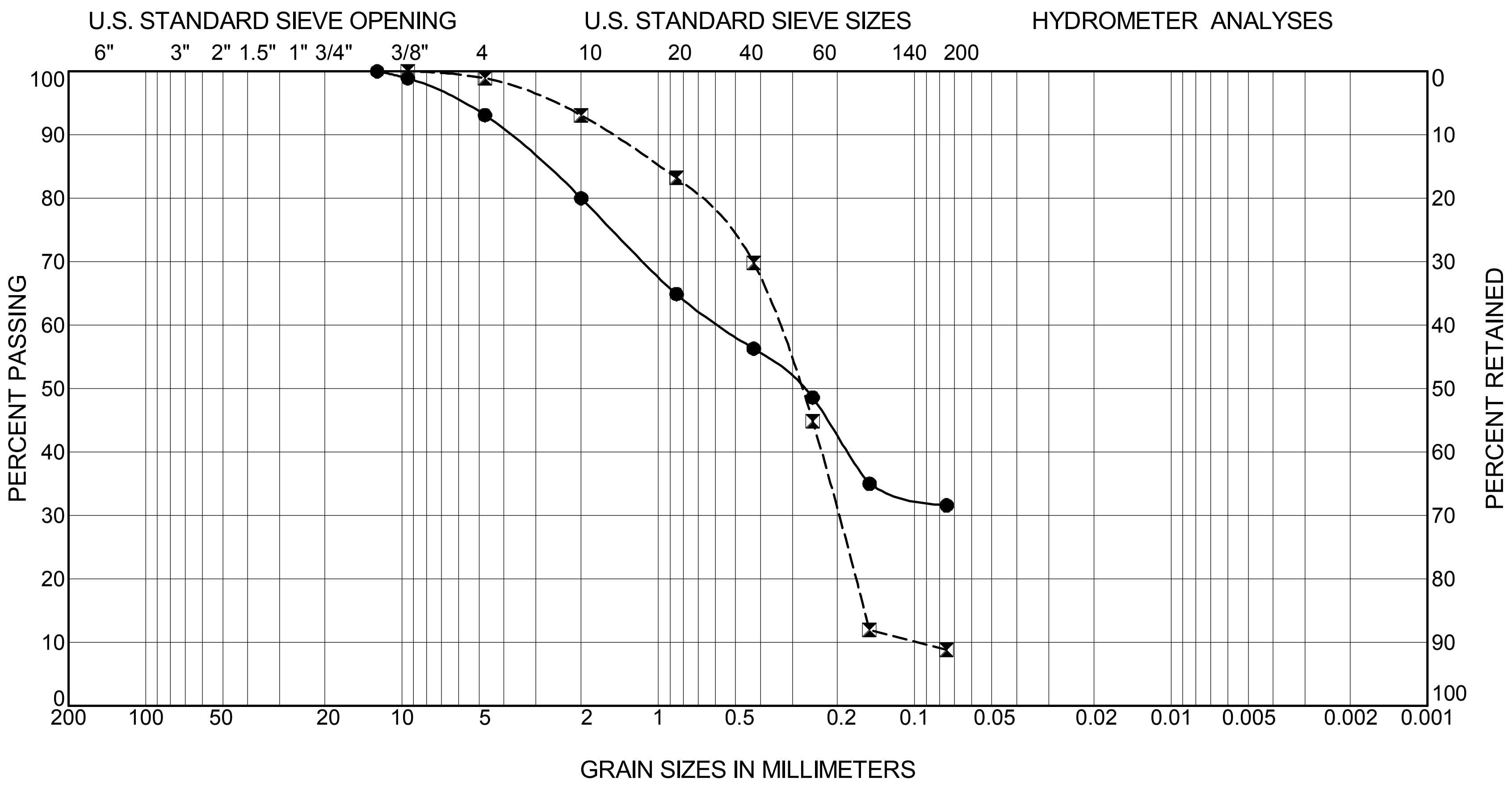
PLATE NO: B-3A

GRAIN SIZE DISTRIBUTION CURVES

COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Description
MK-P-1	MC-4	14.5	●			CLAYEY SAND (SC)
MK-P-3	MC-8	34.5	⧗			Poorly graded SAND with SILT (SP-SM)

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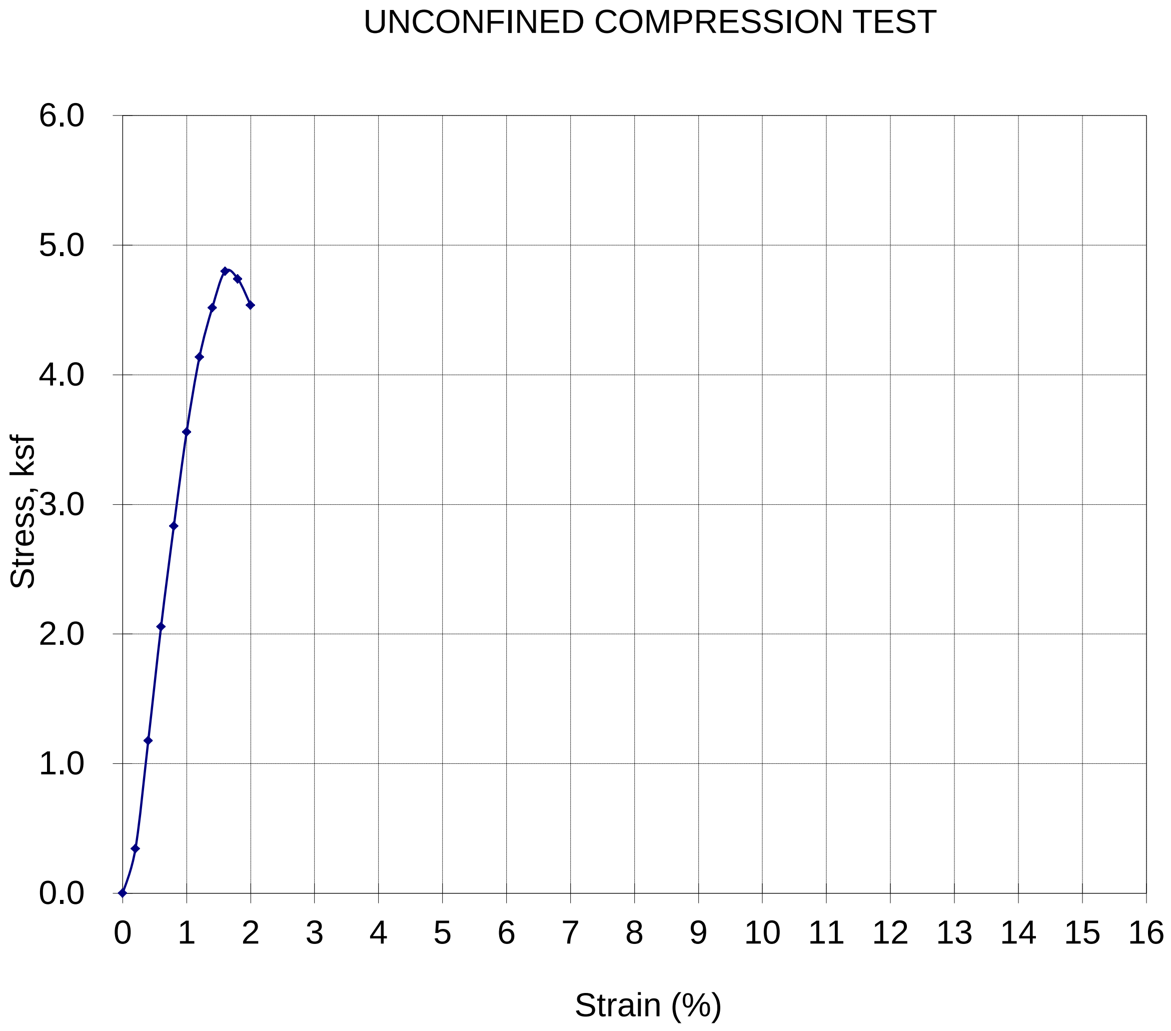
JOB NO: 2010-108-010

PLATE NO: B-3B



Boring No.:	MK-B-1		
Sample No. :	2	**Maximum Strength (ks**	4.80
Depth (feet):	4.5	**Strain @ Failure (%):**	1.60

Material Description:
Hard, Lean Clay with Gravel

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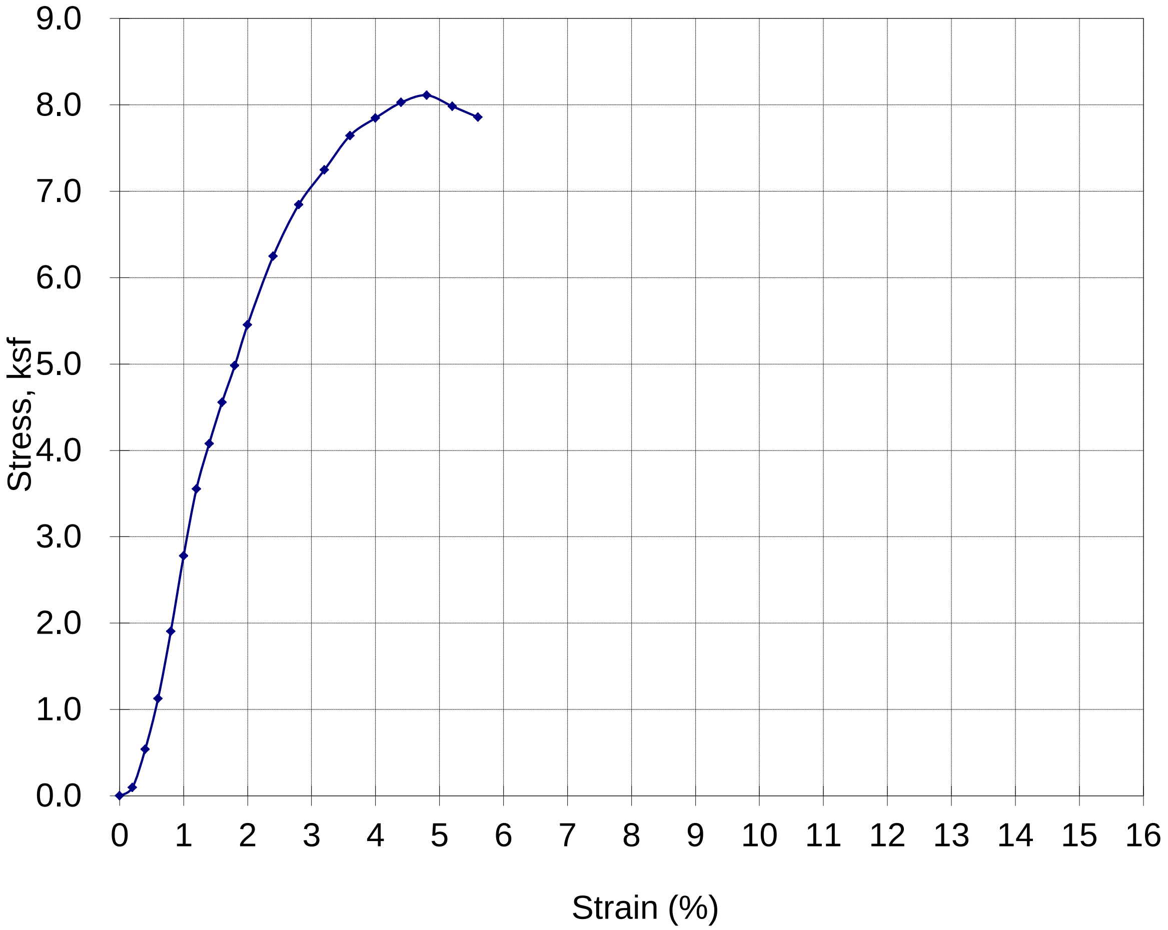
JOB NO.: 2010-108-010 **PLATE NO.: B-4A**

UNCONFINED COMPRESSION TEST



Boring No.:	MK-B-2		
Sample No. :	3	**Maximum Strength (ks**	8.11
Depth (feet):	9.5	**Strain @ Failure (%):**	4.80

Material Description:
Hard, Lean Clay

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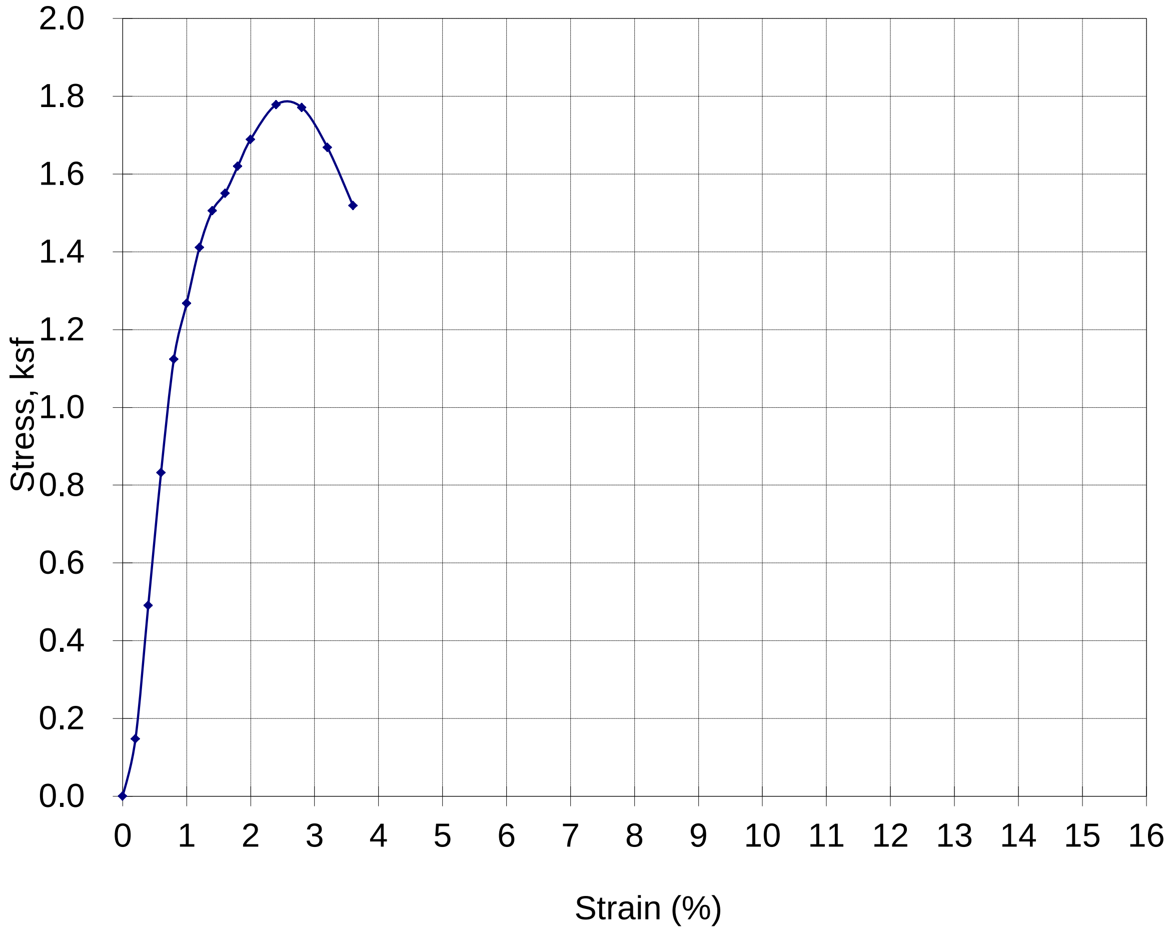
JOB NO.: 2010-108-010 | **PLATE NO.: B-4B**

UNCONFINED COMPRESSION TEST



Boring No.:	MK-B-2		
Sample No. :	7	**Maximum Strength (ks**	1.77
Depth (feet):	29.5	**Strain @ Failure (%):**	2.80

Material Description:
Firm, Silt with Sand

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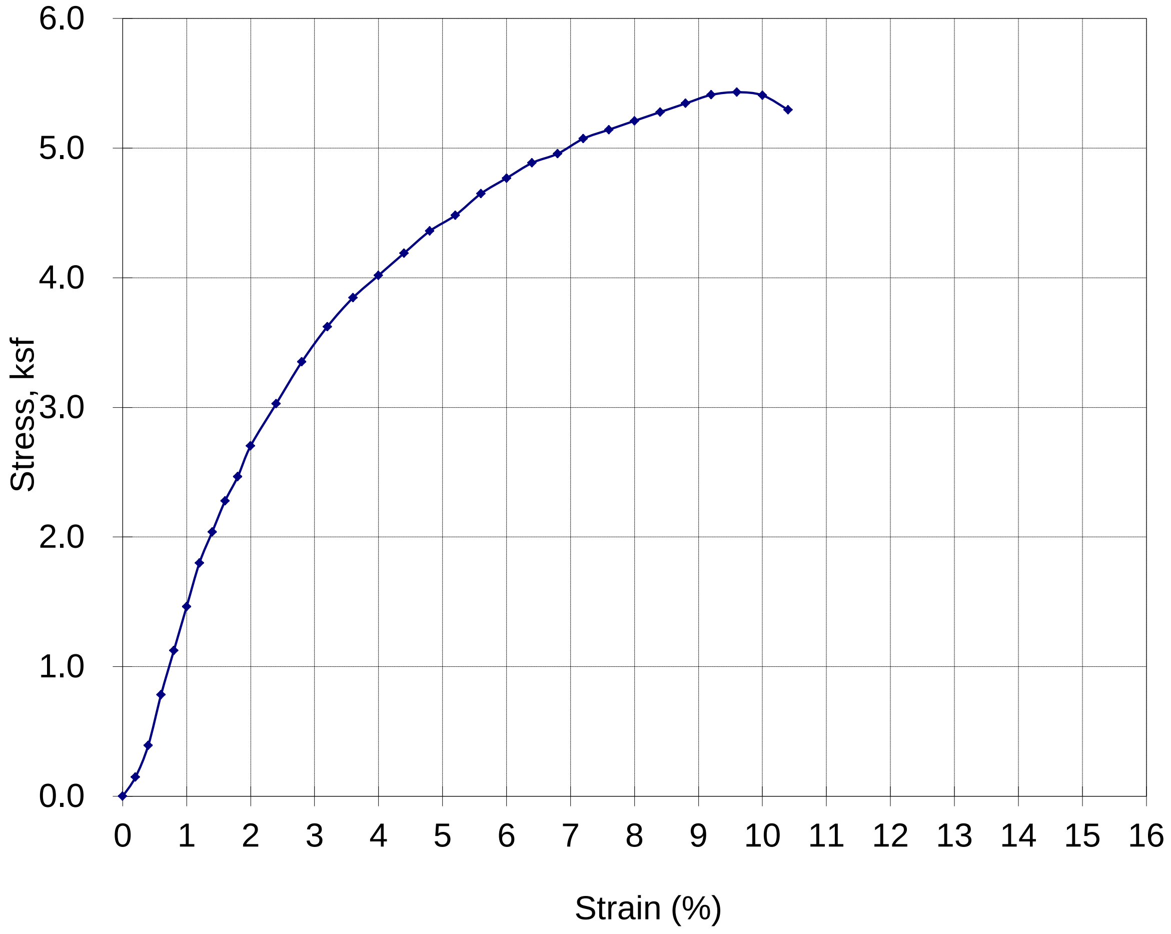
JOB NO.: 2010-108-010	**PLATE NO.: B-4C**

UNCONFINED COMPRESSION TEST



Boring No.: MK-B-3

Sample No. : 6 **Maximum Strength (ks** 5.43

Depth (feet): 24.5 **Strain @ Failure (%):** 9.60

Material Description:
Hard, Lean Clay

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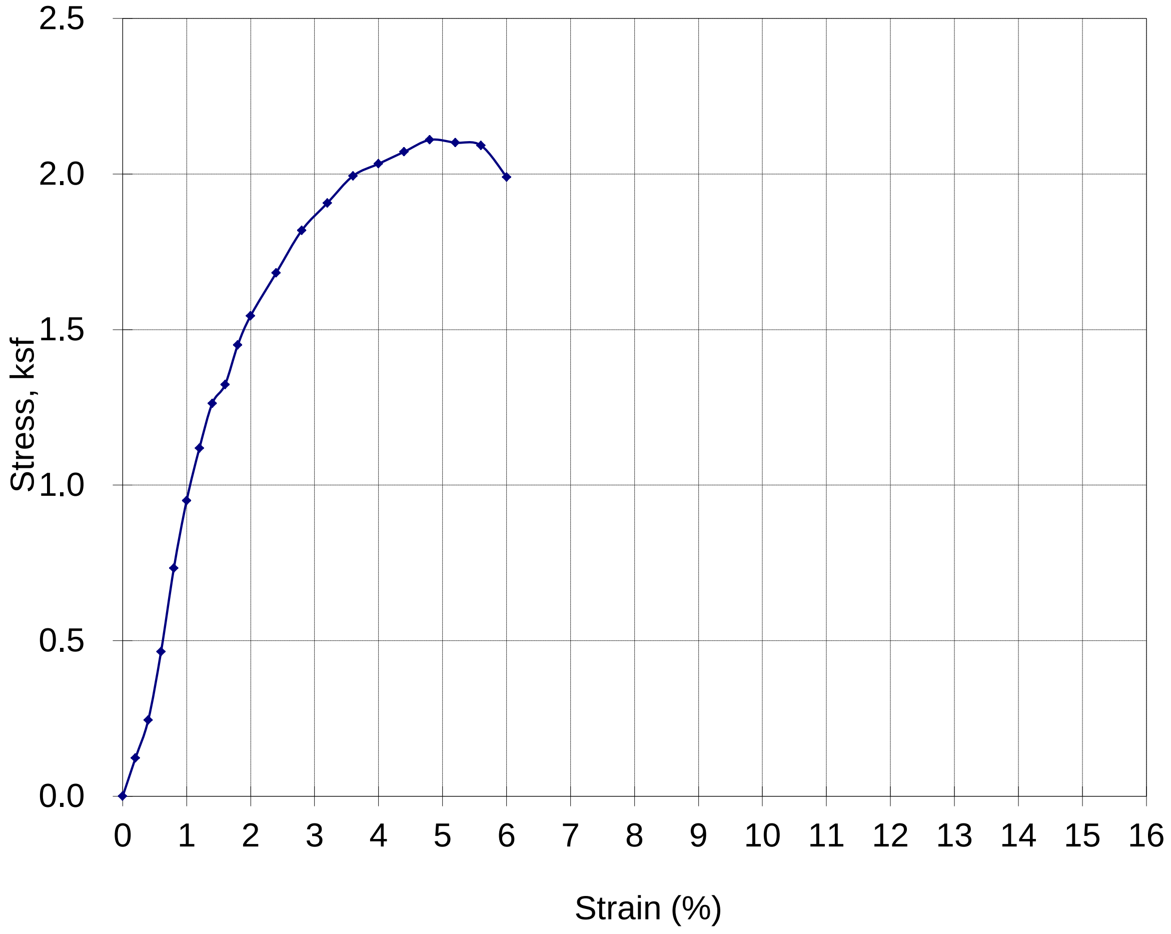
JOB NO.: 2010-108-010 **PLATE NO.: B-4D**

UNCONFINED COMPRESSION TEST



Boring No.: MK-B-4

Sample No. : 3 **Maximum Strength (ks** 2.11

Depth (feet): 8 **Strain @ Failure (%):** 4.80

Material Description:
Stiff, Lean Clay with Gravel

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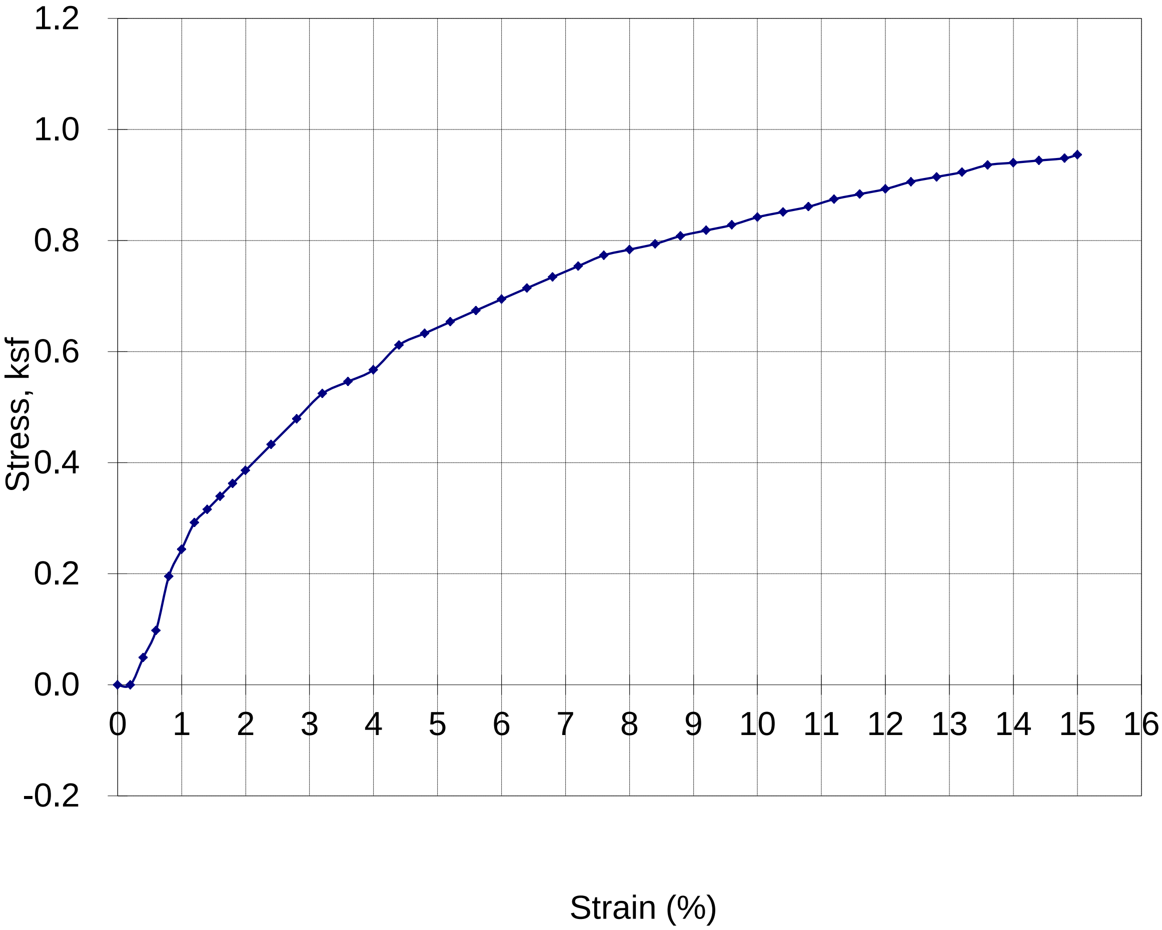
JOB NO.: 2010-108-010 **PLATE NO.: B-4E**

UNCONFINED COMPRESSION TEST



Boring No.: MK-B-5

Sample No. : 1 **Strength (ksf):** 0.95

Depth (feet): 2 **Strain (%):** 15.00

Material Description:
Soft, Sandy Lean Clay

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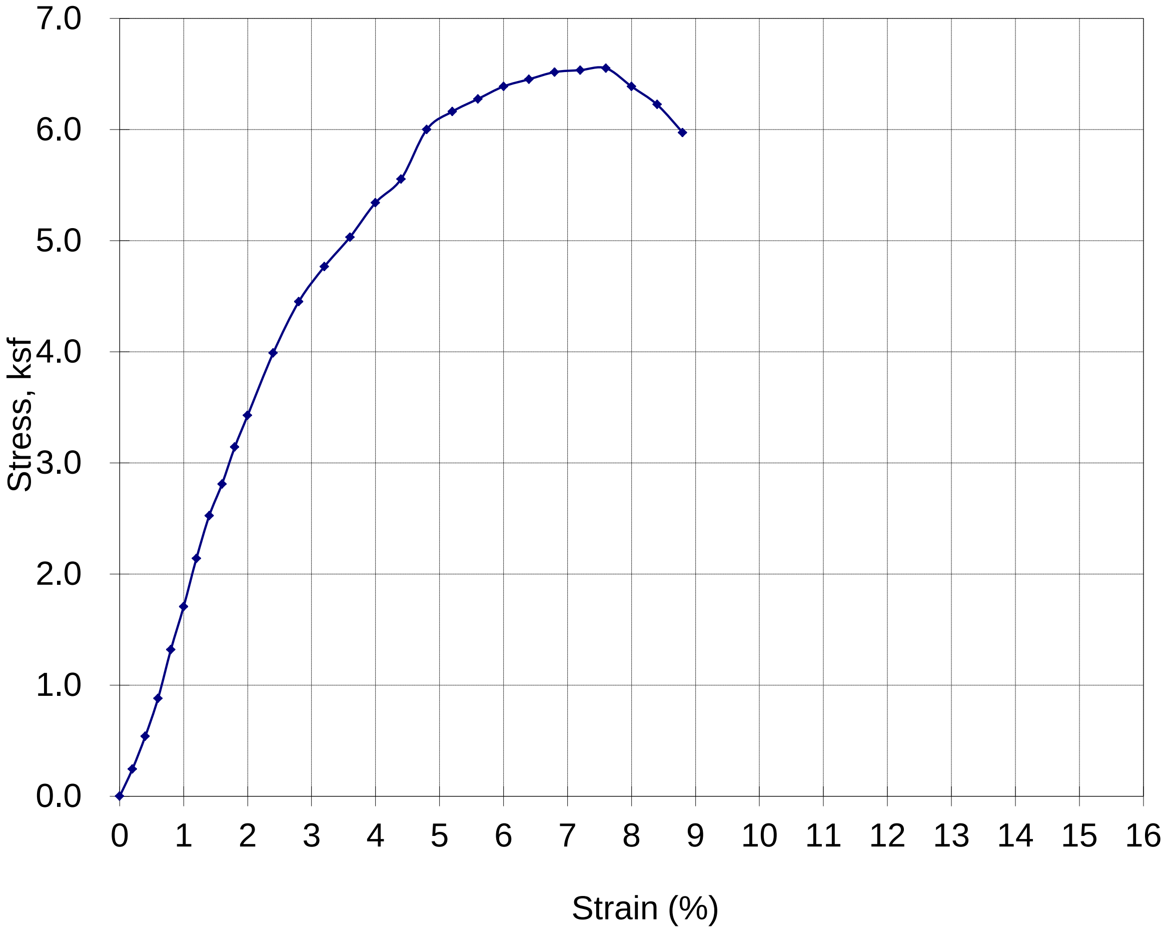
JOB NO.: 2010-108-010 **PLATE NO.: B-4F**

UNCONFINED COMPRESSION TEST



Boring No.:	MK-B-6		
Sample No. :	2	**Maximum Strength (ks**	6.55
Depth (feet):	4.5	**Strain @ Failure (%):**	7.60

Material Description:
Hard, Lean Clay

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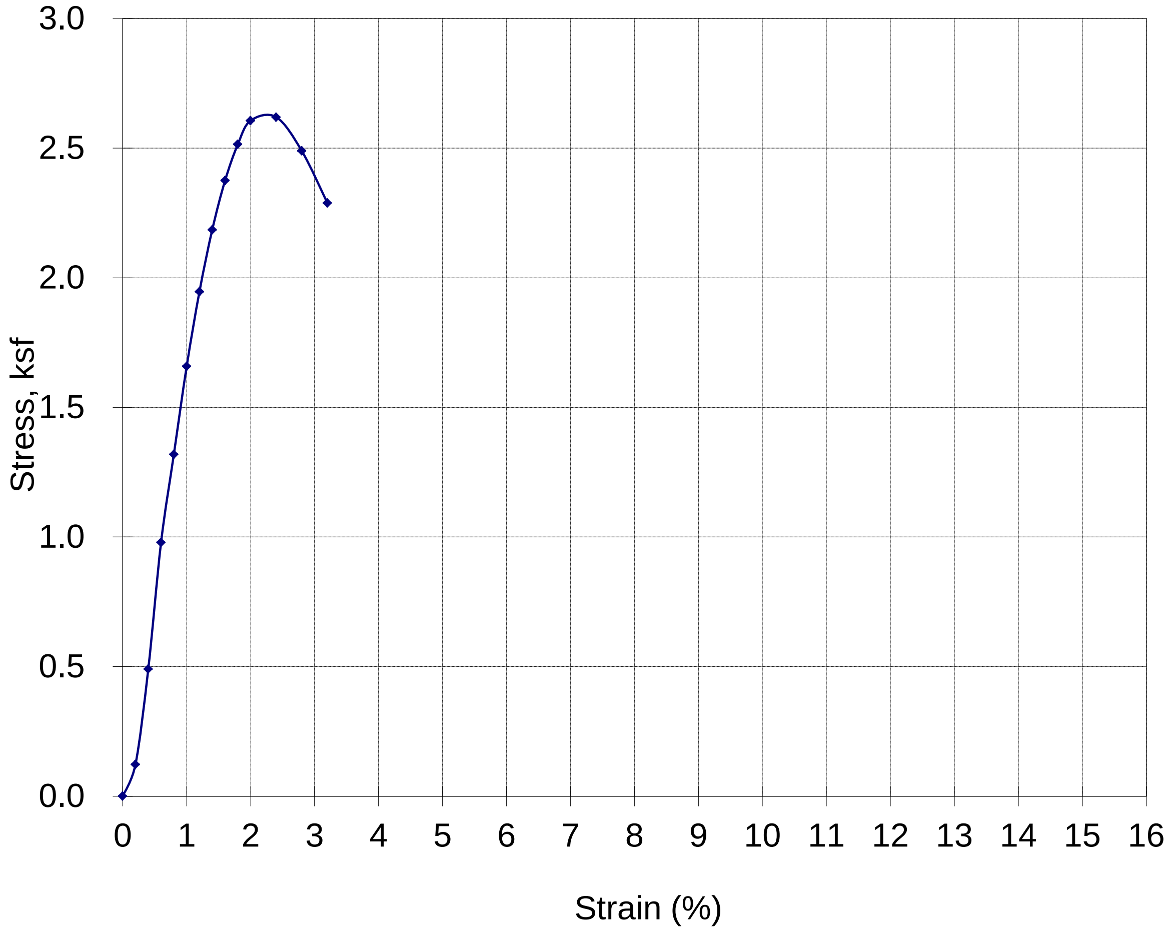
JOB NO.: 2010-108-010	**PLATE NO.: B-4G**

UNCONFINED COMPRESSION TEST



Boring No.: MK-P-2

Sample No. : 4 **Maximum Strength (ks** 2.62

Depth (feet): 14.5 **Strain @ Failure (%):** 2.40

Material Description:
Stiff, Lean Clay

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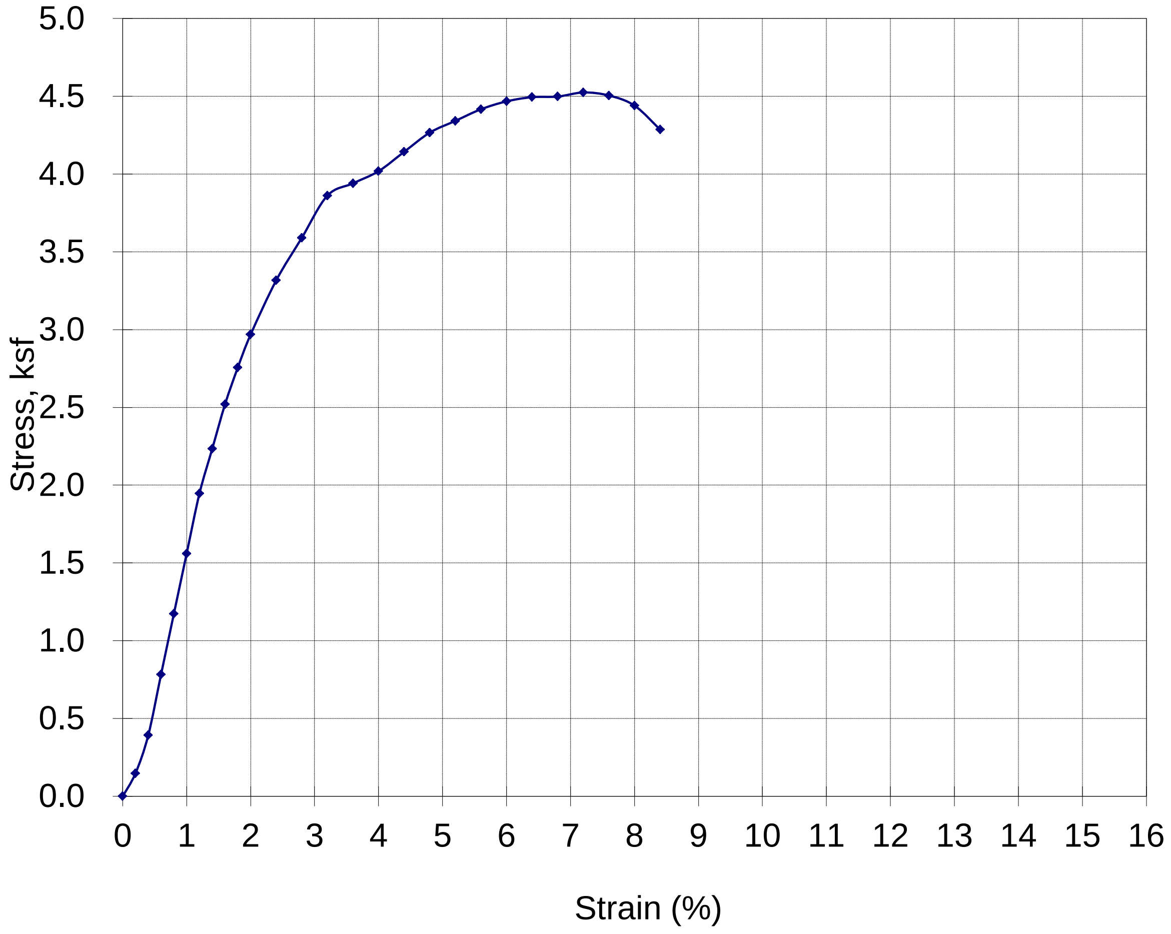
JOB NO.: 2010-108-010	**PLATE NO.: B-4H**

UNCONFINED COMPRESSION TEST



Boring No.: MK-P-2

Sample No. : 9 **Maximum Strength (ks** 4.52

Depth (feet): 39.5 **Strain @ Failure (%):** 7.20

Material Description:
Hard, Lean Clay

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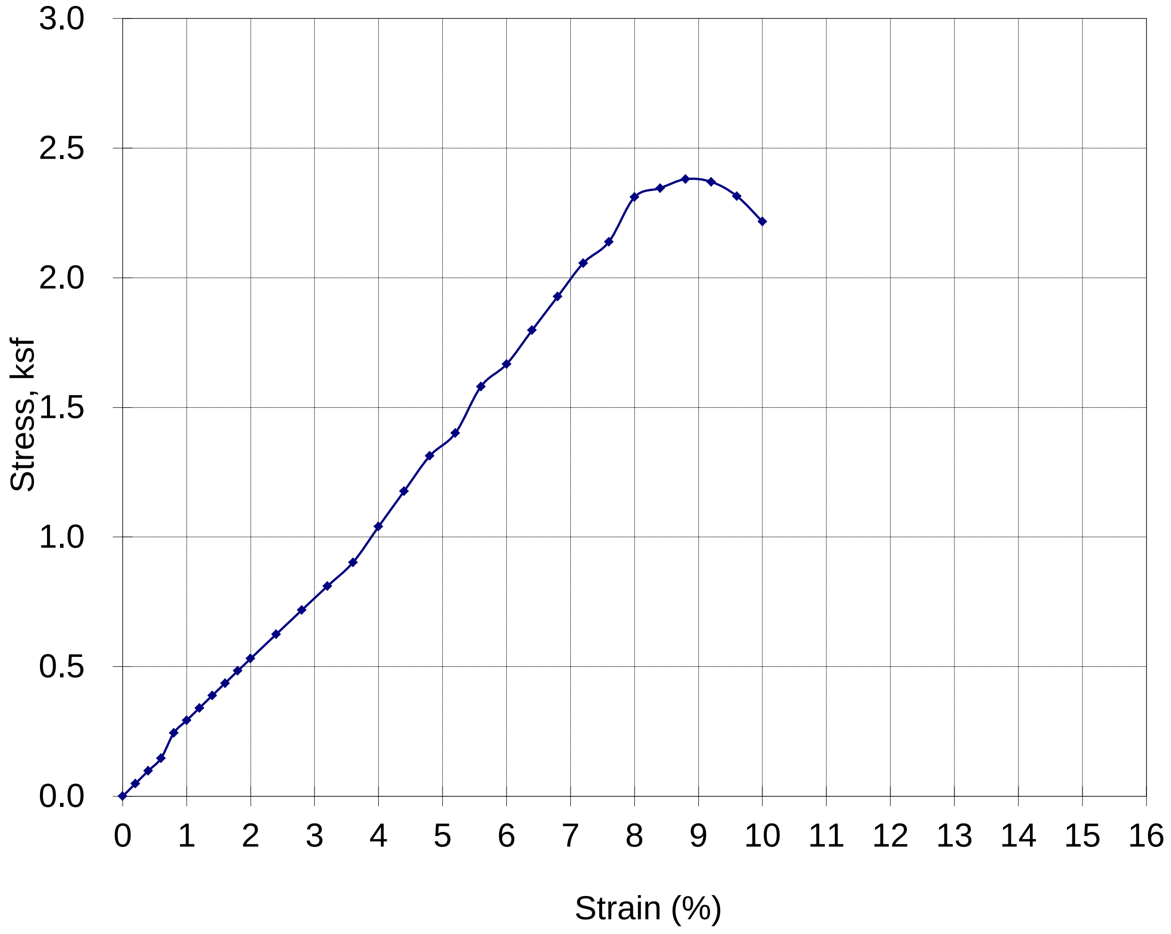
JOB NO.: 2010-108-010	**PLATE NO.: B-4I**

UNCONFINED COMPRESSION TEST



Boring No.:	MK-P-4		
Sample No. :	7	**Maximum Strength (ks**	2.38
Depth (feet):	29.5	**Strain @ Failure (%):**	8.80

Material Description:
Stiff, Silt

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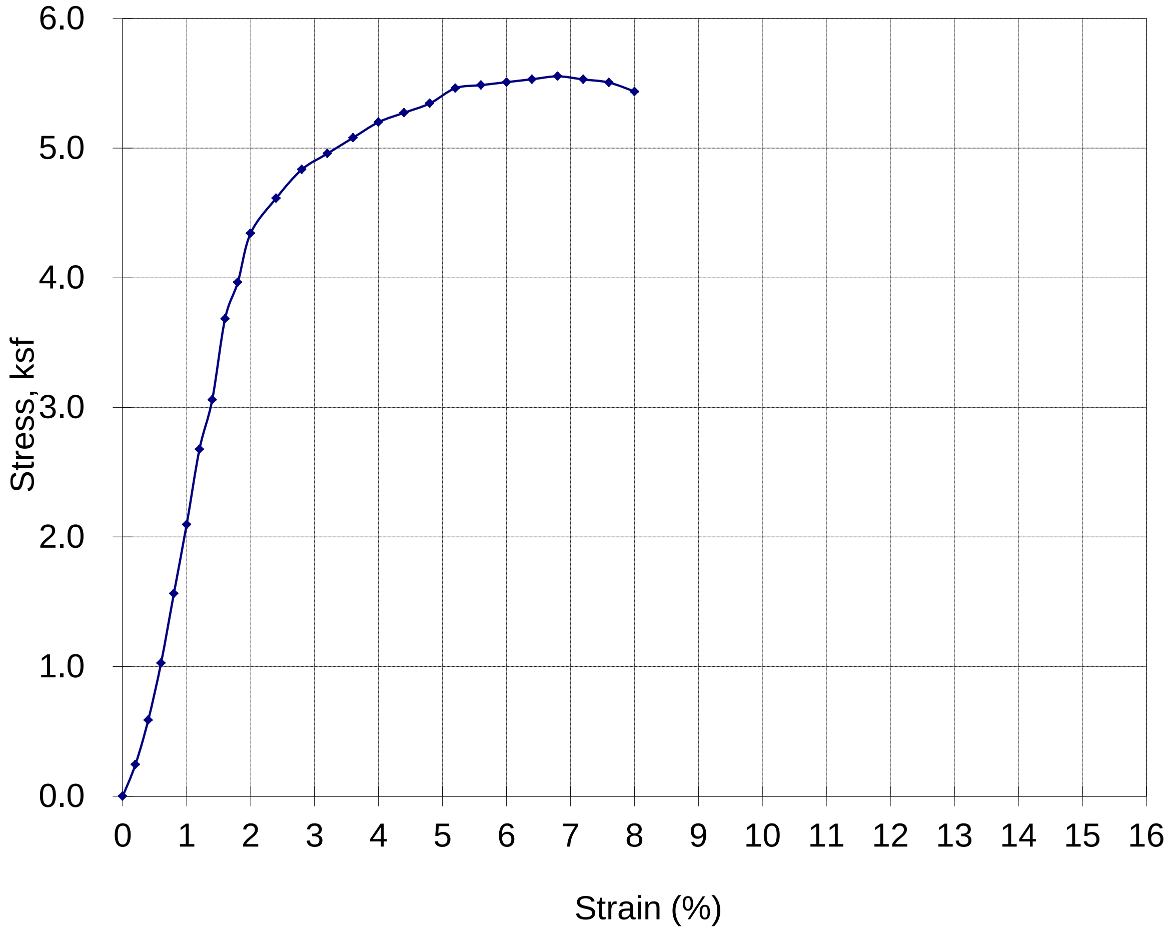
JOB NO.: 2010-108-010	**PLATE NO.: B-4J**

UNCONFINED COMPRESSION TEST



Boring No.:	MK-P-4		
Sample No. :	8	**Maximum Strength (ks**	5.50
Depth (feet):	34.5	**Strain @ Failure (%):**	9.80

Material Description:
Hard, Lean Clay

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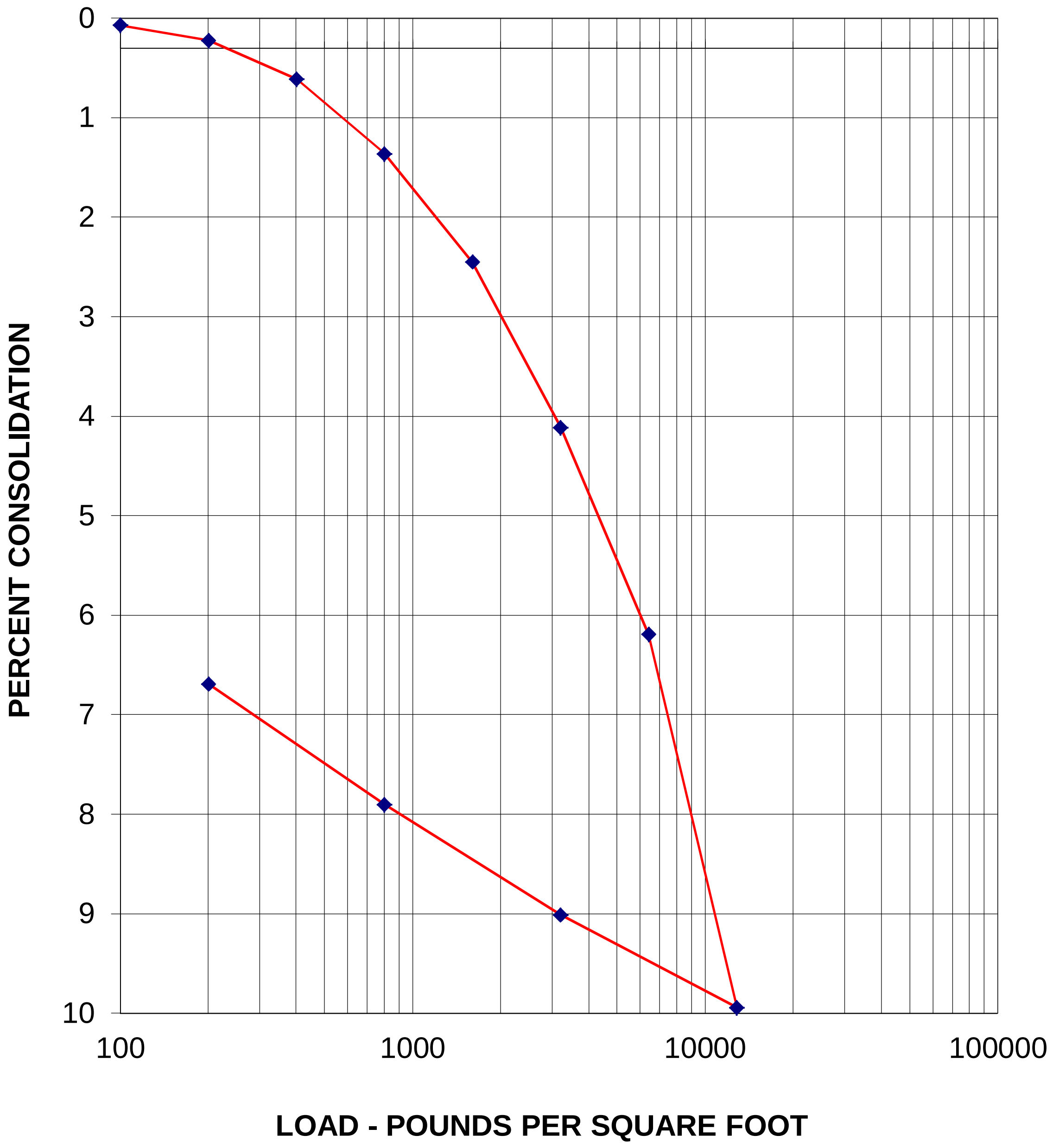
JOB NO.: 2010-108-010	**PLATE NO.: B-4K**

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	21.0	107.6	1.0000	2.416
FINAL	19.2	115.3	0.9331	2.416

BORING NO.	MK-B-5	SAMPLE NO.	3	ELEV. OR DEPTH	9.5'
DESCRIPTION	Sandy Lean Clay, reddish brown				

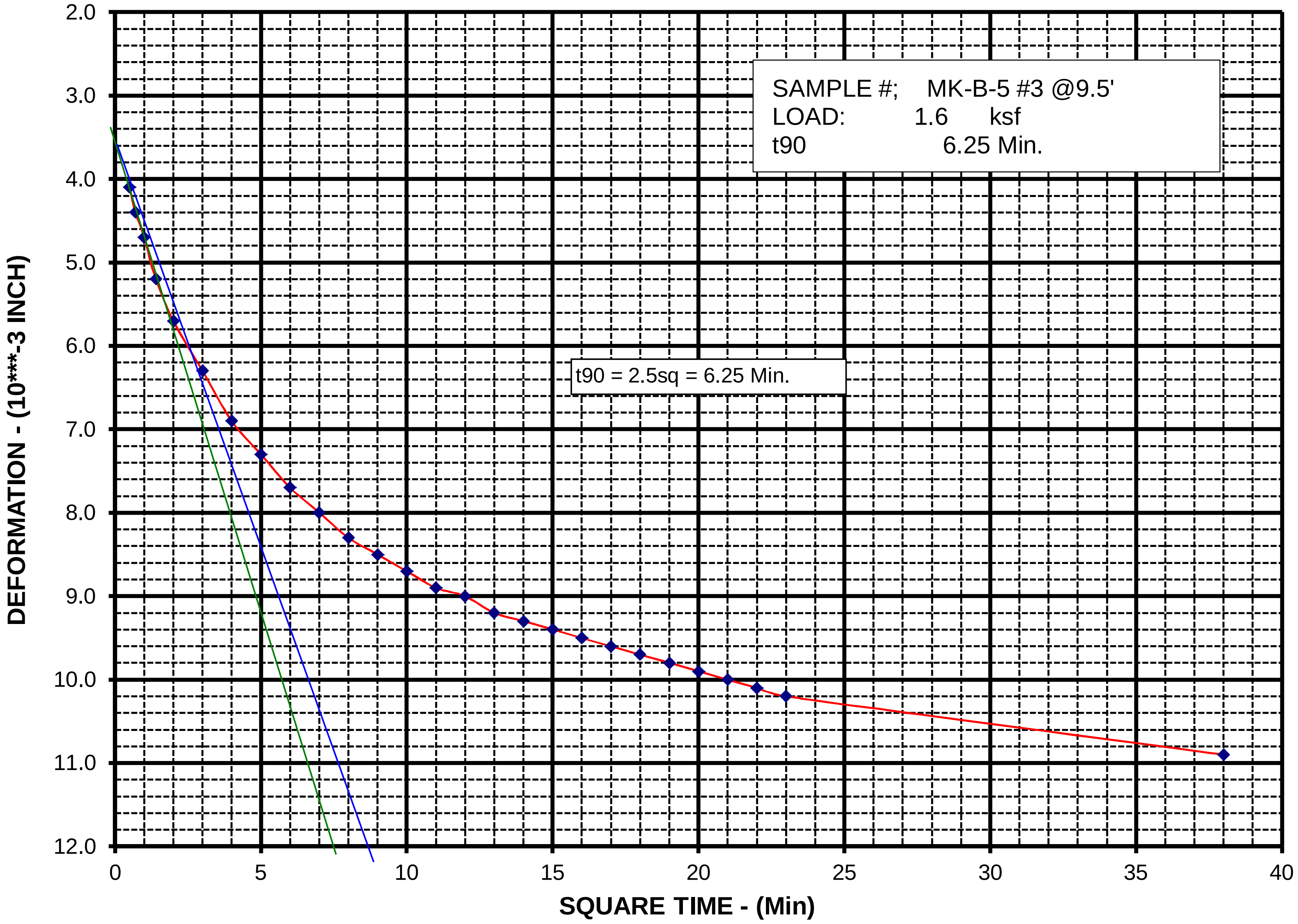
PARIKH
Practicing in the Geosciences

Permanente Creek Flood Protection Project
Flood Protection Improvements - McKelvey Park
Mountain View, California

Date: 6/15/10		Job No.: 2010-108-010	
Reported by: Prav Dayah		Plate No.: 5A	

CONSOLIDATION TEST - TIME CURVE

JOB #: 2010-108-010


SAMPLE #; MK-B-5 #3 @9.5'
LOAD: 1.6 ksf
t90 6.25 Min.

t90 = 2.5sq = 6.25 Min.

Plate No.: 5B

CONSOLIDATION TEST - TIME CURVE

JOB #: 2010-108-010

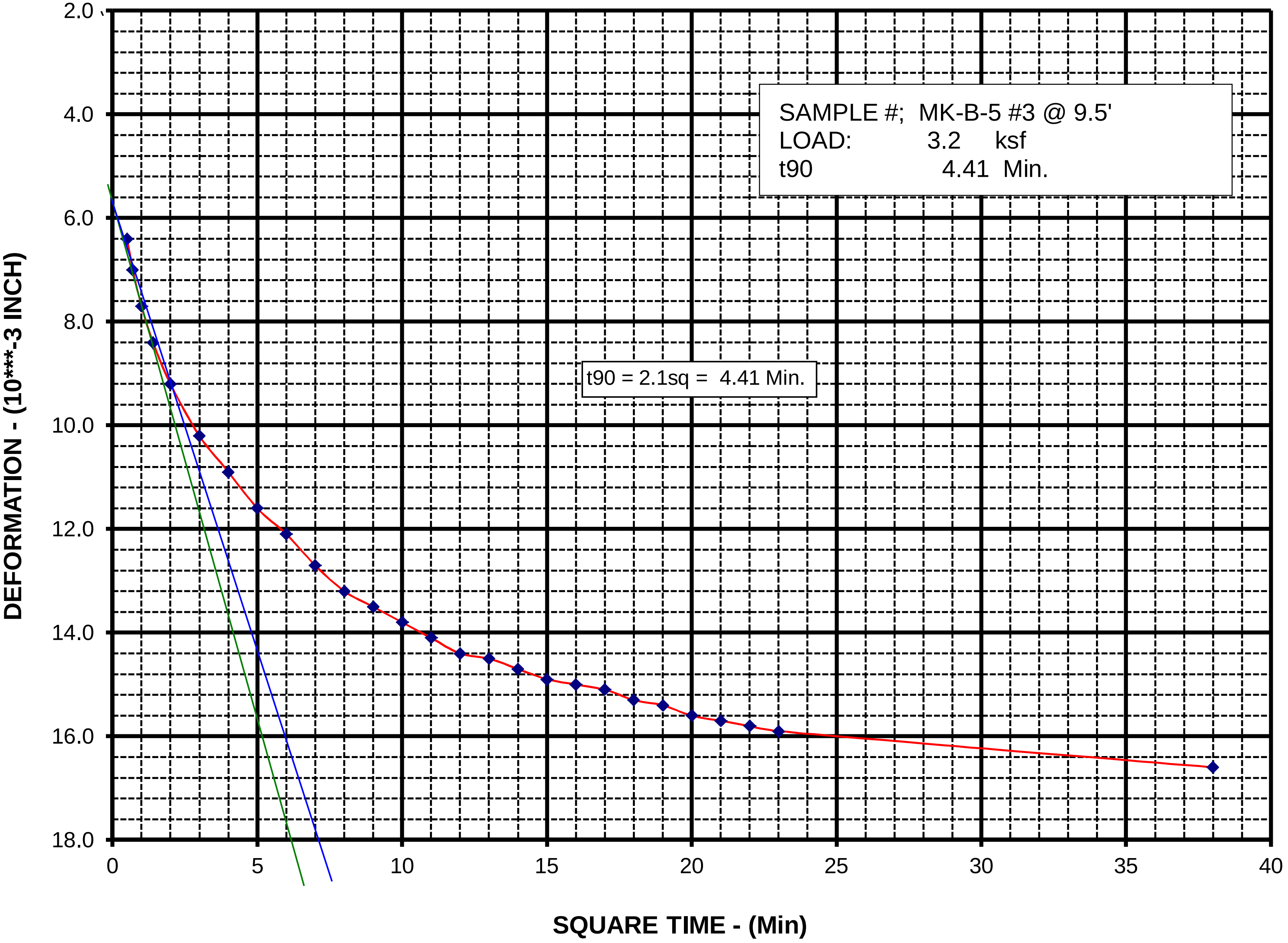


Plate No.: 5C

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 06/23/2010
Date Submitted 06/17/2010

To: Prav Dayah
Parikh Consultants, Inc.
2360 Qume Dr, Ste.A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2010-108-010 /PERMANENTE Site ID : MK-B-5#4 @ 14.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 58096-117902.

EVALUATION FOR SOIL CORROSION

Soil pH	7.78	
Minimum Resistivity	2.36 ohm-cm (x1000)	
Chloride	4.3 ppm	00.00043 %
Sulfate	4.4 ppm	00.00044 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
JUN 25 2010

PLATE NO.: 6A

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 07/14/2010
Date Submitted 07/09/2010

To: Prav Dayah
Parikh Consultants, Inc.
2360 Qume Dr, Ste.A
San Jose, CA 95131

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 2010-108-010/ PERMANENTE Site ID : MK-P2#3@9.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 58280-118314.

EVALUATION FOR SOIL CORROSION

Soil pH	7.75	
Minimum Resistivity	1.69 ohm-cm (x1000)	
Chloride	36.9 ppm	00.00369 %
Sulfate	49.6 ppm	00.00496 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

RECEIVED
JUL 16 2010
By

PLATE NO.: 6B

APPENDIX C

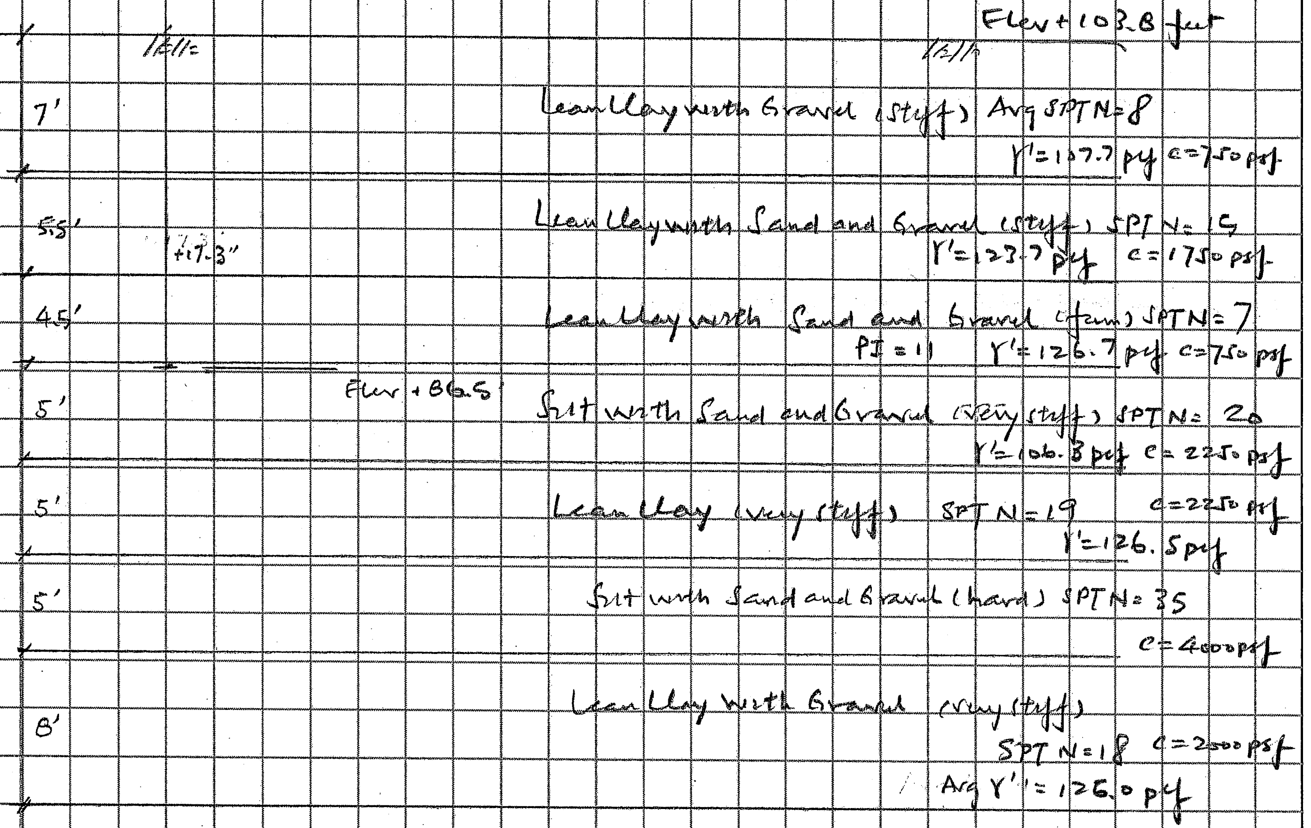
Bearing Capacity Analyses

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT Bearing Capacity Analysis

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Pa
CALCULATED BY A Lam DATE 2/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

Generalized Soil Profile (Boring MK-P-2) - Retaining Wall No. 1. Max height = 22'
(Toe Pressure = 2.76 ksf)



Lean Clay with Gravel (stiff) Avg SPT N=8
γ'=117.7 pcf c=750 psf

Lean Clay with Sand and Gravel (stiff) SPT N=15
γ'=123.7 pcf c=1750 psf

Lean Clay with Sand and Gravel (firm) SPT N=7
PI=11 γ'=126.7 pcf c=750 psf

Silt with Sand and Gravel (very stiff) SPT N= 20
γ'=106.8 pcf c=2250 psf

Lean Clay (very stiff) SPT N=19 c=2250 psf
γ'=126.5 pcf

Silt with Sand and Gravel (hard) SPT N=35
c=4000 psf

Lean Clay with Gravel (very stiff)
SPT N=18 c=2500 psf
Avg γ'=126.0 pcf

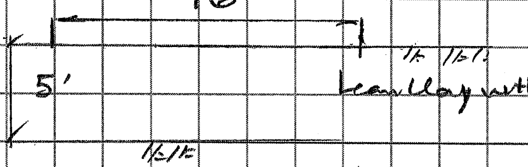
Groundwater was not encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - Mckelvey Park
CALCULATED BY Alam DATE 5/2014
CHECKED BY ____ DATE ____
VERIFIED BY ____ DATE ____
BACK CHECKED BY ____ DATE ____

SUBJECT Retaining Wall No. 1

Bearing Capacity Analyses - (Boring MK-P-2)

Assume



Lean Clay with Sand and Gravel (SPT N=7) c = 900 psf

Silt with Sand and Gravel (SPT N=20) c = 2250 psf

DM.7 - Figure 5

c_1 = 850 psf c_2 = 2250 psf c_2/c_1 = 2.5

T = 5', B = 16' T/B = 0.31 N_c = 10

D/B = 0 N_{cD}/N_c = 1.0 N_{cD} = 10

$Q_{ult} = c_1 N_{cD} + \gamma D$

$= 850 \times 10 + (130 - 62.4) \times 2.0 \times 1.0$

= 8635.21 psf Say $\underline{\underline{8.6 \text{ ksf}}}$

$Q_{all} = \frac{8635.2 \text{ psf}}{3.0}$ F.O.S. = 3.0

= 2878.4 psf Say $\underline{\underline{2.9 \text{ ksf}}}$ > 2.76 ksf

SUBJECT Bearing Capacity Analyses

PROJECT NO. 2010-10B-010
PROJECT NAME Flood Protection Improvements - McKelvey Park
CALCULATED BY A Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

Generalized Soil Profile (Boring MK-B-20W) - Retaining Wall No. 2, Max height = (Toe Pressure = 2.98 ksf) 24'

Elev. ~ +108.8 feet

Lean Clay (Firm to very stiff)

12.5'

Avg SPT N = 12 c = 1250 psf

Avg γ' = 121.7 pcf

Lean Clay with Gravel (very stiff) c = 2750 psf

15' 27.7'

Avg SPT N = 24 γ' = 119.3 pcf

Elev +81.1 feet

Silt with Sand (very stiff) SPT N = 16 c = 1750 psf

5'

γ' = 120.6 pcf

7.5'

Lean Clay with Sand and Gravel (very stiff) c = 2750 psf

Avg SPT N = 23 γ' = 130.4 pcf

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - Mckelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

SUBJECT Retaining Wall No. 2

Bearing Capacity Analyses - (Boring Mk-B-2(W))

Short-term $\phi = 0$ $c = 1750$ psf $B = 17.0$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\gamma' = 125$ pcf

$\phi' = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 1750 + (130 - 62.4) \times 2 \times 1.0$

$= 9812.7$ psf Say $\underline{9.8 \text{ ksf}}$

$Q_{all} = \dfrac{9812.7}{3.0}$ psf Assume F.O.S. = 3.0

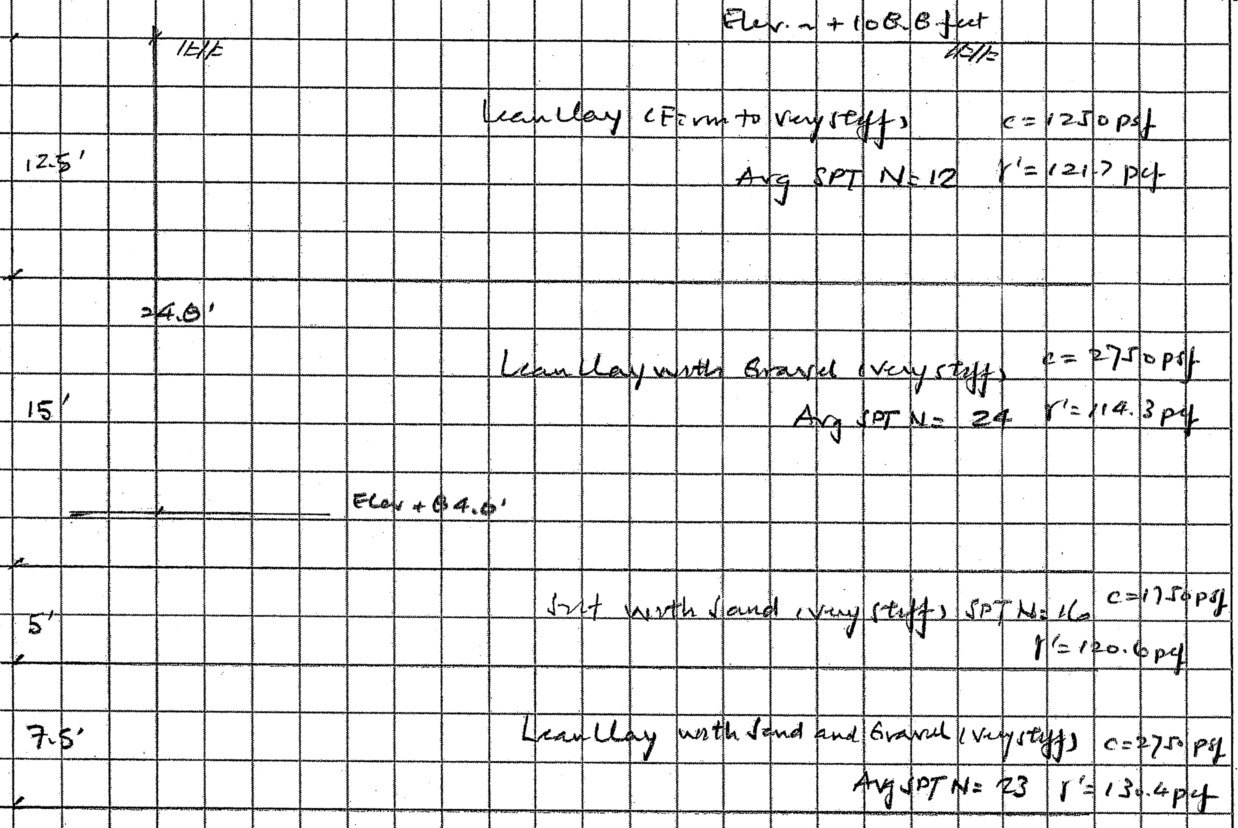
$= 3270.9$ psf Say $\underline{3.3 \text{ ksf}} > 2.98$ ksf

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT Bearing Capacity Analyses

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Park
CALCULATED BY A Lam DATE 2/ 2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

Generalized Soil Profile (Boring MK-B-20W) - Retaining Wall No. 4 Maximum height (Toe Pressure = 2.98 ksf) = 24 feet



Lean Clay (Firm to very stiff) c = 1250 psf
Avg SPT N = 12 $\gamma' = 121.7$ pcf

Lean Clay with Gravel (very stiff) c = 2750 psf
Avg SPT N = 24 $\gamma' = 114.3$ pcf

Silt with Sand (very stiff) SPT N = 16 c = 1750 psf
$\gamma' = 120.6$ pcf

Lean Clay with Sand and Gravel (very stiff) c = 2750 psf
Avg SPT N = 23 $\gamma' = 130.4$ pcf

No groundwater was encountered during drilling

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GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT Retaining Wall No.4

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - McKelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

Bearing Capacity Analysis - (Boring MK-B-2(MW))

Short-term $\phi = 0$ $c = 1750$ psf (assumed) $B = 17$ feet, $D = 2$ feet $\gamma_{sat} = 130$ pcf
$\gamma' = 125$ pcf

$\phi' = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 1750 + (130 - 62.4) \times 2 \times 1.0$

$= 9812.7$ psf say $\underline{\underline{9.8 \text{ ksf}}}$

$Q_{all} = \frac{9812.7 \text{ psf}}{3.0}$ Assume F.o.S. = 3.0

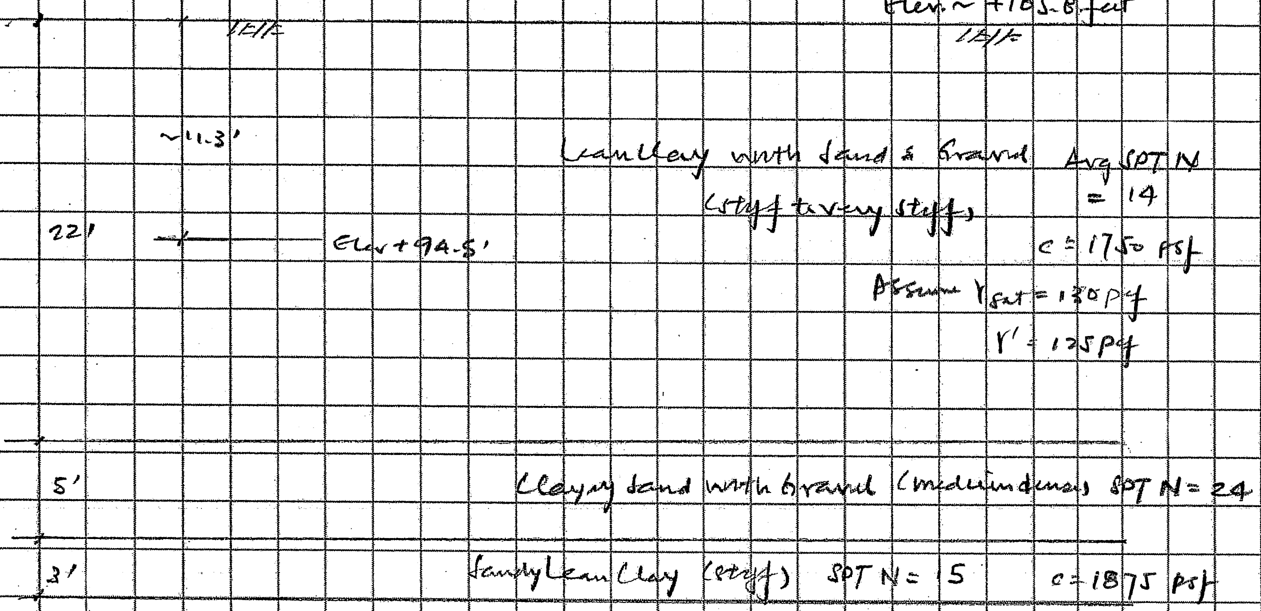
$= 3270.9$ psf say $\underline{\underline{3.3 \text{ ksf}}}$ > 2.98 ksf

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT Bearing Capacity Analyses

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Par
CALCULATED BY A Lam DATE 2/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

Generalized Soil Profile (Boring MK-B-7) - Retaining Wall No. 5 Max height = 10 feet
(Toe Pressure = 1.41 ksf)



Elev ~ +105.8 feet

~11.3'

22' Elev +94.5'

Lean Clay with Sand & Gravel (stiff to very stiff) Avg SPT N = 14

c = 1750 psf

Assume γ_{sat} = 130 pcf

γ' = 125 pcf

5' Clayey Sand with Gravel (medium dense) SPT N = 24

3' Sandy Lean Clay (stiff) SPT N = 15 c = 1875 psf

Groundwater was not encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - Mckelvey Park
CALCULATED BY Alam DATE 5/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

SUBJECT Retaining Wall No. 5

Bearing Capacity Analysis - (Boring Mk-B-7)

Short-term $\phi = 0$ $c = 1750$ psf $B = 9$ feet $D = 2$ feet $\gamma_{sat} = 125$ pcf
$\gamma' = 120$ pcf

$\phi' = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 1750 + (130 - 62.4) \times 2 \times 1.0$

$= 9812.7$ psf Say 9.8 ksf

$Q_{all} = \frac{9812.7}{3.0}$ psf Assume F.O.S. = 3.0

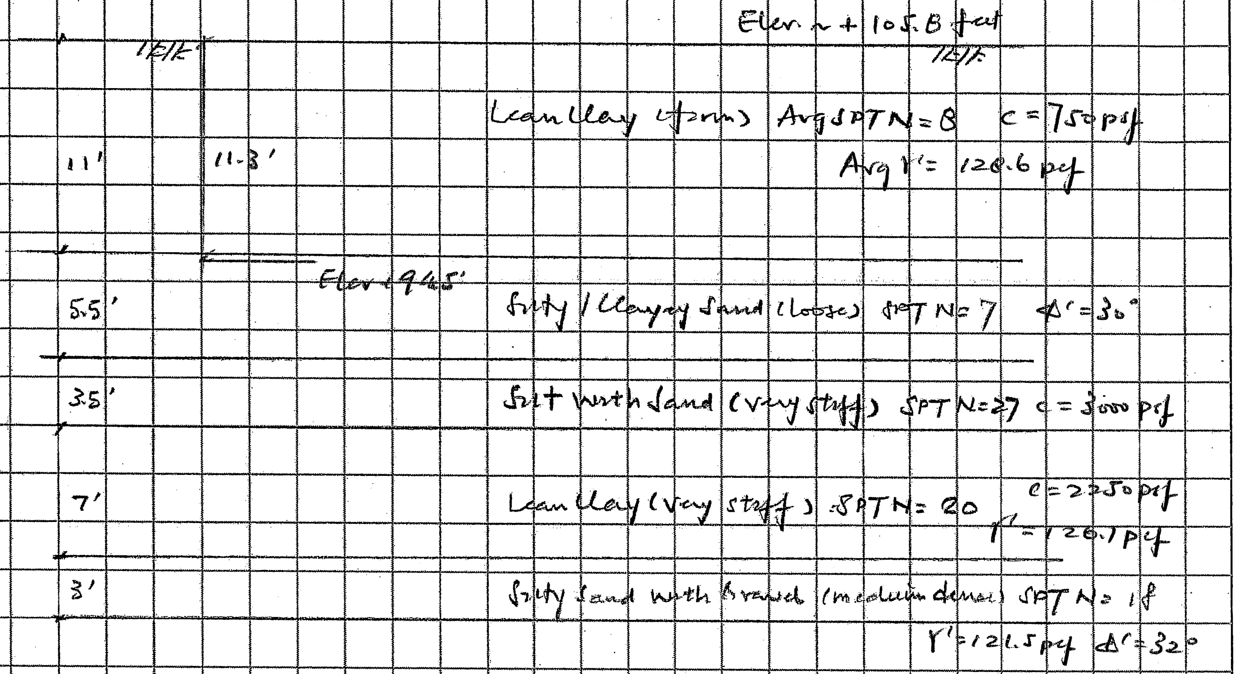
$= 3270.9$ psf Say 3.3 ksf > 1.41 ksf

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

SUBJECT: Bearing Capacity Analyses

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Park
CALCULATED BY A Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

Generalized Soil Profile (Boring MK-B-4) - Retaining Wall No. 6 - Max height = 10 feet
(Toe Pressure = 1.41 ksf)



Lean Clay (firm) Avg SPT N = 8 c = 750 psf
Avg γ' = 120.6 pcf

Silty / Clayey Sand (loose) SPT N = 7 $\phi' = 30°$

Silt with Sand (very stiff) SPT N = 27 c = 3000 psf

Lean Clay (very stiff) SPT N = 20 c = 2250 psf γ' = 126.1 pcf

Silty Sand with Gravel (medium dense) SPT N = 18
γ' = 121.5 pcf $\phi' = 32°$

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - McKelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Retaining Wall No 6

Bearing Capacity Analysis - (Boring MK-B-4)

Short-term $\phi = 0$ $c = 750$ psf $B = 9$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi' = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.50 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 750 + (130 - 62.4) \times 2 \times 1.0$

$= 4282.7$ psf Say $\underline{4.3 \text{ ksf}}$

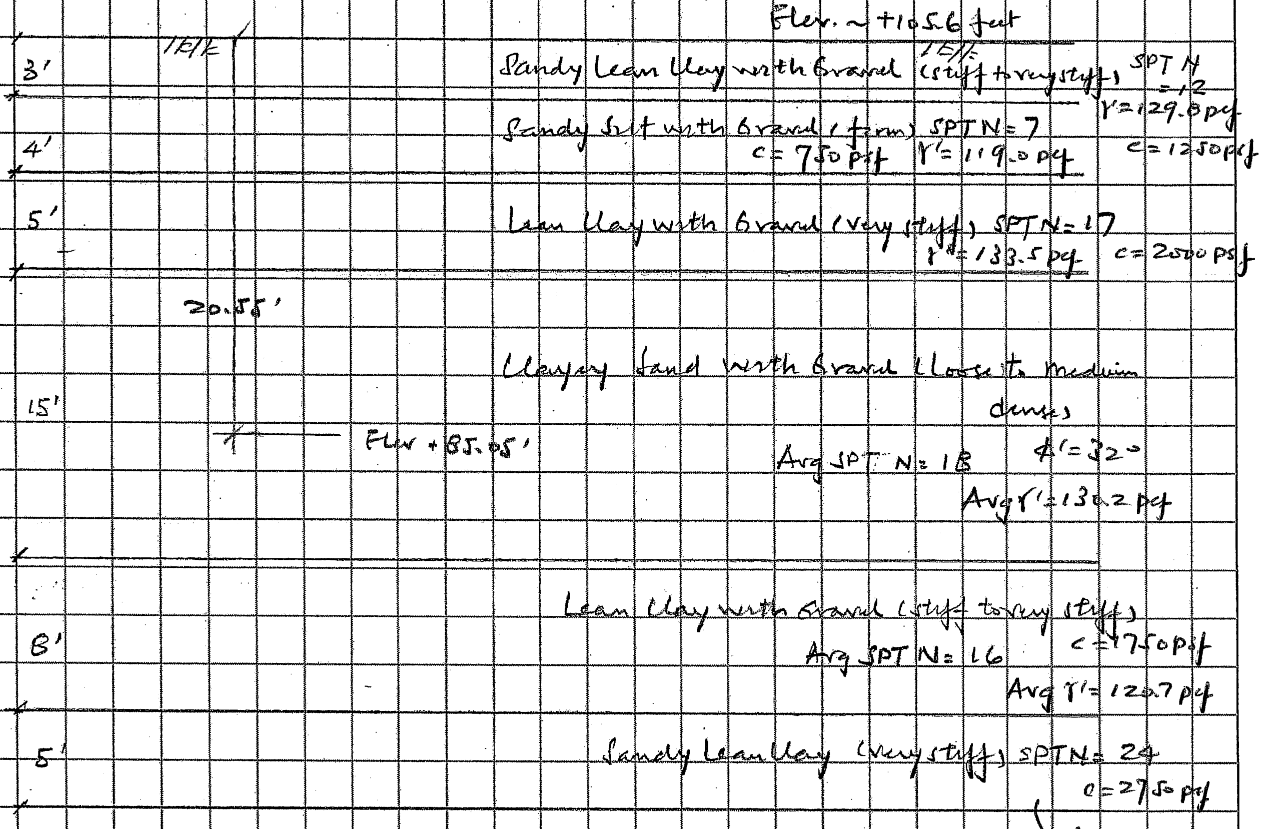
$Q_{all} = \dfrac{4282.7}{3.0}$ psf Assume F.O.S. = 3.0

$= 1427.6$ psf Say $\underline{1.43 \text{ ksf}} > 1.41$ ksf

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - McKelvey Park
CALCULATED BY A Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Bearing Capacity Analyses

Generalized Soil Profile (Boring MK-P-1) - Retaining Wall No. 7 Max height = 24 feet
(Toe Pressure = 2.98 ksf)



3' Sandy Lean Clay with Gravel (stiff to very stiff) SPT N = 12
γ = 129.0 pcf
c = 1250 psf

4' Sandy Silt with Gravel (firm) SPT N = 7
c = 750 psf γ' = 119.0 pcf

5' Lean Clay with Gravel (very stiff) SPT N = 17
γ' = 133.5 pcf c = 2000 psf

15' Clayey Sand with Gravel (loose to medium dense)
Avg SPT N = 18 ϕ' = 32°
Avg γ' = 130.2 pcf

8' Lean Clay with Gravel (stiff to very stiff)
Avg SPT N = 16 c = 1750 psf
Avg γ' = 120.7 pcf

5' Sandy Lean Clay (very stiff) SPT N = 24
c = 2750 psf
γ' = 137.0 pcf

Groundwater was not encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Park
CALCULATED BY ALam DATE 5/2014
CHECKED BY ________ DATE ________
VERIFIED BY ________ DATE ________
BACK CHECKED BY ________ DATE ________

SUBJECT Retaining Wall No 7

Bearing Capacity Analysis - (Boring MK-P-1).

Assume $\phi' = 32°$ $c = 0$ $B = 17$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi' = 32°$, $N_q = 25.0$ $N_c = 38.0$ $N_\gamma = 25.0$ DM7 7.2-131 Figure 1

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 0.5 \times (130 - 62.4) \times 17 \times 25.0 + (130 - 62.4) \times 2 \times 25.0$

$= 14{,}365 + 3{,}380$

$= 17{,}745$ psf Say $\underline{\underline{17.5 \text{ ksf}}}$

$Q_{all} = \dfrac{17{,}745}{3.0}$ psf (Assume F.O.S. = 3.0)

$= 5{,}915$ psf Say $\underline{\underline{5.9 \text{ ksf}}} > 2.98$ ksf

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - Mckelvey Park
CALCULATED BY A Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Bearing Capacity Analyses

Generalized Soil Profile (Boring MK-B-20W) Retaining Wall No. 8 Max height = 24 feet
(Toe Pressure = 2.98 ksf)

Elev. ~ +108.8 feet

Lean Clay (Firm to very stiff)
12.5'
Avg SPT N=12 c = 1250 psf
Avg γ' = 121.7 pcf

Lean Clay with Gravel (very stiff) c = 2750 psf
15' ~+25.5'
Avg SPT N = 24 γ' = 114.3 pcf

Elev + 83.34

Silt with Sand (very stiff) SPT N= 16 c = 1750 psf
5'
γ' = 120.6 pcf

Lean Clay with Sand and Gravel (very stiff) c = 2750 psf
7.5'
Avg SPT N = 23 γ' = 130.4 pcf

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - Mckelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

SUBJECT Retaining Wall No. 8

Bearing Capacity Analyses - (Boring Mt-B-21x)

Short-term $\phi = 0$ $c = 1750$ psf (assumed) $B = 17$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 1750 + (130 - 62.4) \times 2 \times 1.0$

$= 9812.7$ psf Say $\underline{\underline{9.8 \text{ ksf}}}$

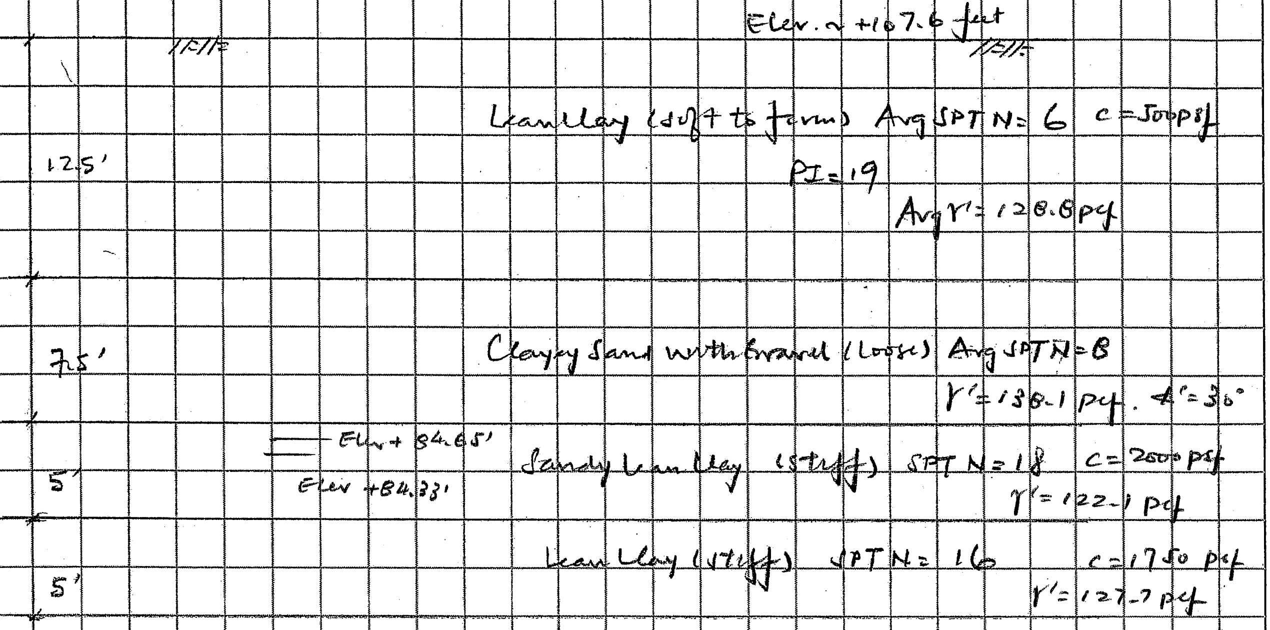
$Q_{all} = \dfrac{9812.7}{3.0}$ Assume F.O.S. = 3.0

$= 3270.9$ psf Say $\underline{\underline{3.3 \text{ ksf}}} > 2.98$ ksf

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - McKelvey Park
CALCULATED BY A Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Bearing Capacity Analyses

Generalized Soil Profile (Boring MK-B-5) - Retaining Wall No. 9 Max height = 20 feet
Retaining Wall No. 10 Max height = 16 feet
(20 feet Toe Pressure = 2.73 ksf)
(16 feet Toe Pressure = 2.44 ksf)



Lean Clay (soft to firm) Avg SPT N = 6 c = 500 psf
PI = 19
Avg γ' = 128.8 pcf

Clayey Sand with Gravel (Loose) Avg SPT N = 8
γ' = 130.1 pcf. ϕ' = 30°

Sandy Lean Clay (stiff) SPT N = 18 c = 2000 psf
γ' = 122.1 pcf

Lean Clay (stiff) SPT N = 16 c = 1750 psf
γ' = 127.7 pcf

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - McKelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY ____ DATE ____
VERIFIED BY ____ DATE ____
BACK CHECKED BY ____ DATE ____

SUBJECT Retaining Wall No.9 & No.10

Bearing Capacity Analysis - (Boring MK-B-5)

Short-term $\phi=0$ $c = 2000$ psf $B = 13$ feet $D = 2.0$ feet $\gamma_{sat} = 130$ pcf

$\phi=0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 2000 + (130 - 62.4) \times 2 \times 1.0$

$= 11195.2$ psf Say $\underline{\underline{11.2 \text{ ksf}}}$

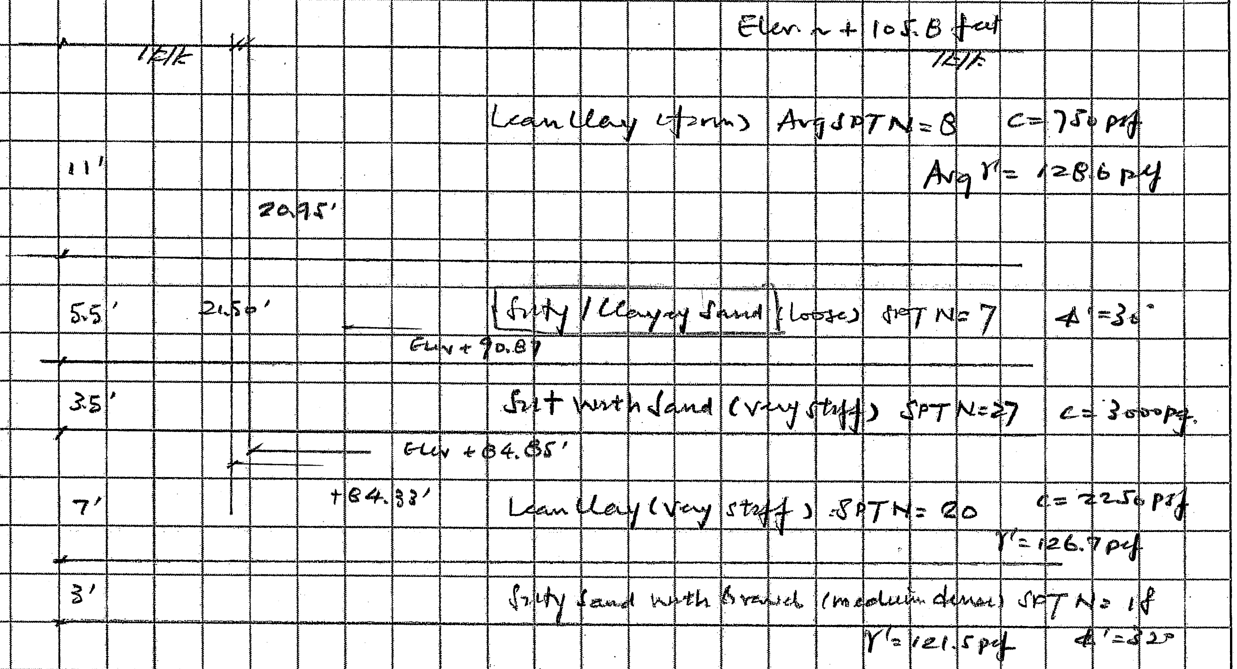
$Q_{all} = \dfrac{11195.2}{3.0}$ Assume F.O.S. = 3.0

$= 3731.7$ psf Say $\underline{\underline{3.7 \text{ ksf}}} > 2.73$ ksf

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - McKeeley Park
CALCULATED BY Alam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Bearing Capacity Analyses

Generalized Soil Profile (Boring MK-B-4) - Retaining Wall No. 11 Max height = 14 feet



Lean Clay (firm) Avg SPT N = 8 c = 750 psf
Avg γ' = 128.6 pcf

Silty / Clayey Sand (loose) SPT N = 7 ϕ' = 30°

Silt with Sand (very stiff) SPT N = 27 c = 3000 psf

Lean Clay (very stiff) SPT N = 20 c = 2250 psf
γ' = 126.7 pcf

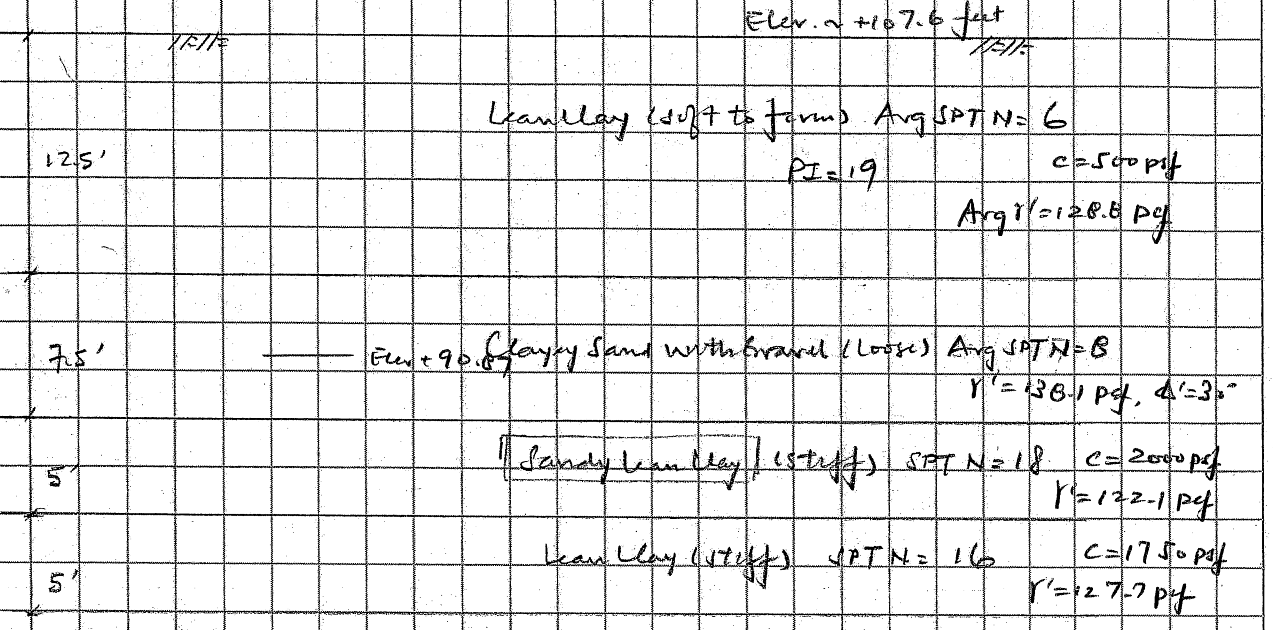
Silty Sand with Gravel (medium dense) SPT N = 18
γ' = 121.5 pcf ϕ' = 32°

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvements - McKelvey Park
CALCULATED BY A. Lam DATE 2/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Bearing Capacity Analysis

Generalized Soil Profile (Boring MK-B-5) Retaining Wall No. 11 Max height = 14 feet
Toe Pressure = 1.95 ksf)



Elev. ~ +107.6 feet

Lean Clay (soft to firm) Avg SPT N = 6
PI = 19
c = 500 psf
Avg γ' = 128.8 pcf

Elev. +90.8 Clayey Sand with Gravel (Loose) Avg SPT N = 8
γ' = 138.1 pcf, ϕ' = 30°

Sandy Lean Clay (stiff) SPT N = 18 c = 2000 psf
γ' = 122.1 pcf

Lean Clay (stiff) SPT N = 16 c = 1750 psf
γ' = 127.7 pcf

No groundwater was encountered during drilling

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - Mckelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Retaining Wall No 11

Bearing Capacity Analyses - (Boring MK-B-5)

Assume $\phi = 30^\circ$ $c = 0$ $B = 10.5$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi' = 30^\circ$ $N_q = 20.0$ $N_c = 33.0$ $N_\gamma = 17.0$ DM7 7.2-131 Figure 1

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 0.5 \times (130 - 62.4) \times 10.5 \times 20.0 + (130 - 62.4) \times 2 \times 17.0$

$= 9396.4$ psf say 9.4 ksf

$Q_{all} = \frac{9396.4 \text{ psf}}{3.0}$ Assume F.O.S. = 3.0

$= 3132.1$ psf say 3.1 ksf > 1.95 ksf

Assume $\phi = 0$ $c = 2000$ psf $B = 10.5$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi = 0$ $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 2000 + (130 - 62.4) \times 2.0 \times 1.0$

$= 11195.2$ psf say 11.2 ksf

$Q_{all} = \frac{11195.2}{3.0}$

$= 3731.7$ psf say 3.7 ksf

PROJECT NO. 2016-108-010
PROJECT NAME Flood Protection Improvements - McKelvey Par
CALCULATED BY A Lam DATE 2/2014
CHECKED BY DATE
VERIFIED BY DATE
BACK CHECKED BY DATE

SUBJECT Bearing Capacity Analyses

Generalized Soil Profile - (Boring MK-B-3) - Retaining Wall No. 2 Max height = 20 feet
(Toe Pressure = 2.73 ksf)

Elev. ~ +107.2 feet

1E/1F

3' — 1E/1E Lean Clay with Sand (stiff) SPT SPT N=12 c=1250 psf, $\gamma'=132.6$ pcf

4.5' — Clayey Sand with Gravel (very loose) SPT N=4 $\phi'=28°$, $\gamma'=123.4$ pcf

4.5' — Silt with Sand (hard) SPT N= 35 c= 4000 psf

5' — Lean Clay with Sand (firm to stiff) SPT N=11 c= 1000 psf, PI= 16, $\gamma'=129.1$ pcf

5.5' — Sandy lean Clay (stiff) SPT N= 13 c= 1500 psf, $\gamma'= 115.5$ pcf

Elev + 83.5 feet

7.5' — Lean Clay (very stiff) Avg SPT N= 15 c=1750 psf, Avg $\gamma'=126.6$ pcf

No groundwater was encountered during drilling.

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

PROJECT NO. 2010-108-010
PROJECT NAME Flood Protection Improvement - McKelvey Park
CALCULATED BY A Lam DATE 5/2014
CHECKED BY ______ DATE ______
VERIFIED BY ______ DATE ______
BACK CHECKED BY ______ DATE ______

SUBJECT Retaining Wall No. 12

Bearing Capacity Analyses - (Boring MK-B-3)

Short-term $\phi = 0$ $c = 1500$ psf $B = 16$ feet $D = 2$ feet $\gamma_{sat} = 130$ pcf

$\phi = 0$, $N_c = 5.53$ $N_q = 1.0$ $N_\gamma = 0$

$Q_{ult} = 0.5 \times \gamma B N_\gamma + c N_c + \gamma D N_q$

$= 5.53 \times 1500 + (130 - 62.4) \times 2 \times 1.0$

$= 8430.2$ psf Say $\underline{\underline{8.4 \text{ ksf}}}$

$Q_{all} = \dfrac{8430.2 \text{ psf}}{3.0}$ Assume F.O.S. = 3.0

$= 2810.0$ psf Say $\underline{\underline{2.8 \text{ ksf}}} > 2.73$ ksf